NEW ZEALAND

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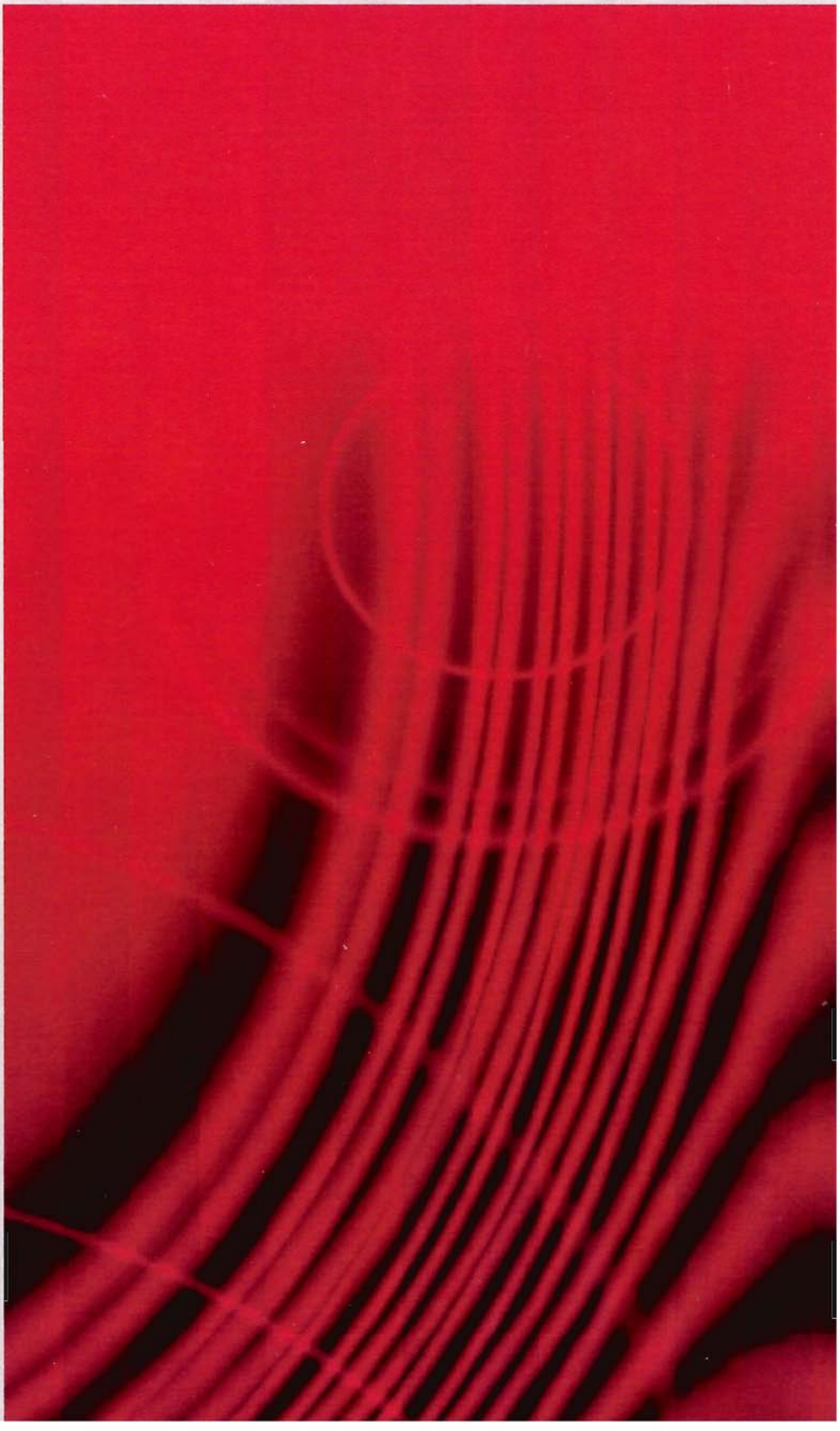
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PAEREWA AOTEAROA

NZS 3101:Part 1:2006

Incorporating Amendment No. 1 & 2

Concrete Structures Standard

NZS 3101:Part 1:2006



New Zealand Standard

CONCRETE STRUCTURES STANDARD

Part 1 –
The Design of Concrete Structures

ISBN 1-86975-043-8

COMMITTEE REPRESENTATION

This Standard was prepared by the Concrete Design Committee P 3101 for the Standards Council established under the Standards Act 1988.

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(b) The assistance provided by Stefano Pampanin for work on Appendix B; and

(c) The American Concrete Institute for permission to use extracts from ACI 318-02, Building Code Requirements for Reinforced Concrete. Appendix CF contains specific information related to ACI 318 provisions.

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1	July 2006	Corrects minor errors	SNZ July 2006
2	August 2008	Amends clauses, equations, figures, notations, referenced documents and tables	SNZ August 2008

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Part 5:2005	Earthquake actions – New Zealand
NZS 3104:2003	Specification for concrete production
NZS 3106:1986	Code of practice for concrete structures for the storage of liquids
NZS 3109:1997	Specification for concrete construction
NZS 3112:- - - -	Methods of test for concrete
Part 1:1986	Tests relating to fresh concrete
Part 2:1986	Tests relating to the determination of strength of concrete
NZS 3113:1979	Specification for chemical admixtures for concrete
NZS 3121:1986	Specification for water and aggregate for concrete
NZS 3122:1995	Specification for Portland and blended cements
NZS 3152:1974	Specification for the manufacture and use of
(R) 1980	structural and insulating lightweight concrete
NZS 3404:- - - -	Steel structures standard
Part 1:1997	Steel structures standard

JOINT AUSTRALIA/NEW ZEALAND STANDARDS

AS/NZS 1170:- - - -	Structural design actions
Part 0: 2002	General principles
Part 1: 2002	Permanent, imposed and other actions
Part 2: 2003	Wind actions
Part 3: 2003	Snow and ice actions
AS/NZS 1554:- - - -	Structural steel welding
Part 3:2002	Welding of reinforcing steel
AS/NZS 2699:- - - -	Built-in components for masonry construction
Part 3:2002	Lintels and shelf angles (durability requirements)
AS/NZS 3582:	Supplementary cementitious materials for use with Portland and blended cement
Part 3:2002	Amorphous silica
AS/NZS 4548:1999	Guide to long-life coatings for concrete and masonry
AS/NZS 4671:2001	Steel reinforcing materials
A2 AS/NZS 4672:- - - -	Steel prestressing materials
Part 1:2007	General requirements
AS/NZS 4680:1999	Hot-dip galvanised (zinc) coatings on fabricated ferrous articles

AMERICAN STANDARDS

American Concrete Institute

ACI 210R-93	Erosion of Concrete in Hydraulic Structures (reapproved 1998)
ACI 210.1R-94	Compendium of case histories on repair of erosion-damaged concrete in hydraulic structures (reapproved 1999)
ACI 318-02	Building code requirements for structural concrete
ACI 355.2-01	Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete

American Society for Testing and Materials

ASTM C512-02	Standard test method for creep of concrete in compression
ASTM C1152-04	Standard test method for acid-soluble chloride in mortar and concrete

AUSTRALIAN STANDARDS

AS 1012:- - - -	Methods of testing concrete
Part 10-2000	Determination of indirect tensile strength of concrete cylinders ("Brazil" or splitting test)
Part 11-2000	Determination of the modulus of rupture
Part 13-1992	Determination of the drying shrinkage of concrete for samples prepared in the field or in the laboratory
Part 16-1996	Determination of creep of concrete cylinders in compression
Part 20-1992	Determination of chloride and sulfate in hardened concrete and concrete aggregates
AS 1214-1983	Hot-dip galvanised coatings on threaded fasteners (ISO metric coarse thread series)
AS 1310-1987	Steel wire for tendons in prestressed concrete
AS 1311-1987	Steel tendons for prestressed concrete – 7-wire stress-relieved steel strand for tendons in prestressed concrete
AS 1313-1989	Steel tendons for prestressed concrete – Cold-worked high-tensile alloy steel bars for prestressed concrete
AS 1478.:- - - -	Chemical admixtures for concrete, mortar and grout
Part 1-2000	Admixtures for concrete
AS 1530:- - - -	Methods for fire tests on building materials, components and structures
Part 4-1997	Fire-resistance test of elements of building construction
AS 3582:- - - -	Supplementary cementitious materials for use with portland and blended cement
Part 1-1998	Fly ash
Part 2-2001	Slag – Ground granulated iron blast-furnace
AS 3600-2001	Concrete structures
AS 4058:1992	Precast concrete pipes (pressure and non-pressure)
AS 4072:- - - -	Components for the protection of openings in fire-resistant separating elements
Part 1-1992	Service penetrations and control joints
AS 4672:- - - -	Steel prestressing materials (in preparation)
AS 5100:- - - -	Bridge design
Part 5:2004	Concrete

BRITISH STANDARDS

BS 476:- - - -	Fire tests on building materials and structures	
Part 20:1987	Method for determination of the fire resistance of elements of construction (general principles)	
Part 21:1987	Methods for determination of the fire resistance of load-bearing elements of construction	
Part 22:1987	Methods for determination of the fire resistance of non-load-bearing elements of construction	
BS 1377:- - - -	Methods of test for soils for civil engineering purposes	A2
Part 3:1990	Chemical and electro-chemical tests	
BS 5400:	Steel, concrete and composite bridges	
Part 10:1980	Code of practice for fatigue	
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BS 8500:- - - -	Concrete. Complementary British Standard to BS EN 206-1	A2
Part 1:2006	Method of specifying and guidance for the specifier	

EUROCODES

prEN 1992:- - - -	Eurocode 2: Design of concrete structures
Part 1.1:2002	General rules. Structural fire design. Revised project team final draft
EN 206:- - - -	Concrete
Part 1:2000	Specification, performance, production and conformity

GERMAN STANDARDS

DIN 4030:- - - -	Assessment of water, soil and gases for their aggressiveness to concrete
Part 2 :1991	Collection and examination of water and soil samples
A2 DIN 4102:- - - -	Fire behaviour of building materials and building components
Part 2:1977	Building components; definitions, requirements and tests

INTERNATIONAL STANDARDS

ISO 834:- - - -	Fire-resistance tests – Elements of building construction
Part 1:1999	General requirements

OTHER PUBLICATIONS

Alkali aggregate reaction: Minimising the risk of damage to concrete: Guidance notes and model specification clauses (Technical Report 3), 2004, Cement & Concrete Association of New Zealand.
Approved Code of Practice for the Safe Handling, Transportation and Erection of Precast Concrete, Occupational Safety and Health Service, Department of Labour, 2002.
Bridge Manual (SP/M/022) second edition, Transit New Zealand, 2003.
New Zealand Building Code Compliance Documents and Handbook, Department of Building and Housing, (formerly the Building Industry Authority), 1992 (as amended up to March 2005).
Creep and Shrinkage in Concrete Bridges, RRU Bulletin 70, Transit New Zealand 1984.
CEB-FIP Model Code 1990

NEW ZEALAND LEGISLATION

Building Act 2004
Standards Act 1988

LATEST REVISIONS

The users of this Standard should ensure that their copies of the above-mentioned New Zealand Standards and referenced overseas Standards are the latest revisions or include the latest amendments. Such amendments are listed in the annual Standards New Zealand *Catalogue* which is supplemented by lists contained in the monthly magazine *Standards* issued free of charge to committee and subscribing members of Standards New Zealand.

FOREWORD

This revision of NZS 3101 has been written with the objective of producing a concrete design standard which is:

(a) Compatible with the loading standards AS/NZS 1170 and NZS 1170.5, and other referenced loading standards;
(b) Intended to provide, in due course (once cited) a verification method for compliance with the New Zealand Building Code;
(c) Organised in component focused sections, for ease of use.

During the revision process, the opportunity has been taken to incorporate various technical advancements and improvements that have been developed since 1995. The non-seismic sections of this Standard are largely based upon ACI 318-02.

The following is a summary of some of the key changes in NZS 3101:

(a) The sections of the standard are component focused rather than force focused;
(b) Summary tables suitable as quick reference guides are provided in the commentary to the sections on beams, columns, walls, and joints;
(c) The expected curvature ductility that can be achieved from the specified detailing has been summarised;
(d) The seismic design philosophy has been made compatible with NZS 1170.5;
(e) Two approaches to capacity design have been included in Appendix D;
(f) The Standard now includes information on Grade 500 reinforcement;
(g) The durability section includes new information for zone C exposure classifications. Information is provided for structures with a specified intended life of 100 years. The durability section has been extended to include guidance on chemical exposure, aggressive soils, abrasion resistance, and fastening protection;
(h) Fire has been amended to include the latest revisions from AS 3600, and guidance is provided on the fire design of thin panel walls that are typically found in warehouse type structures;
(i) An Appendix has been provided on the design of fibre reinforced members;
(j) New provisions have been provided for the structural design of thin panel walls. These include the latest developments in ACI 318 and research results of testing conducted in New Zealand;
(k) A new section has been provided on precast concrete;
(l) The strut and tie method of analysis has been introduced into Part 1 of the Standard. The information is based upon ACI 318-02;
(m) An Appendix has been provided for the design of ductile jointed precast systems.

NOTES

NEW ZEALAND STANDARD

CONCRETE STRUCTURES STANDARD

Part 1 – The Design of Concrete Structures

1 GENERAL

1.1 Scope

1.1.1 *Relationship to NZ Building Code*

1.1.1.1 *Minimum requirements*

This Standard sets out minimum requirements for the design of reinforced and prestressed concrete structures.

This Standard does not cover the design of brittle elements. A brittle element is defined as a structural member that does not satisfy the minimum requirements specified in this Standard. A2

1.1.1.2 *Non Specific Terms*

Where this standard has provisions that are in non-specific or unquantified terms then these do not form part of the verification method for the New Zealand Building Code and the proposed details must be submitted to a building consent authority for approval as part of the building consent application. This includes but is not limited to where the standard calls for special studies, a rational analysis, for engineering judgement to be applied or where the Standard requires tests to be "suitable" or "appropriate".

1.1.2 *Application to bridges*

While this standard has been developed with the intent that it be generally applicable to the design of bridges, and is referenced by the Transit New Zealand Bridge Manual, some aspects are recognised to not be adequately covered by this Standard and designers are advised to make reference to appropriate specialised bridge design technical literature. Aspects of bridge design for which reference to the technical literature should be made include the following:

(a) Design for the combination of shear, torsion and warping in box girders;
(b) Design for deflection control taking into account the effects of creep, shrinkage and differential shrinkage and differential creep;
(c) Design for stress redistribution due to creep and shrinkage;
(d) Design for the effects of temperature change and differential temperature. (Refer to the Transit Bridge Manual for these design actions);
(e) Design for the effects of heat of hydration. This is particularly an issue where thick concrete elements are cast as second stage construction and their thermal movements are restrained by previous construction;
(f) Design for shear and local flexural effects, which may arise where out of plane moments are transmitted to web or slab members, or where the horizontal curvature of post-tensioned cables induces such actions;
(g) Seismic design of piers, where curvature ductility demand (material strain) exceeds the maximum permissible values in 2.6.1.3. A2

1.1.3 *Materials and workmanship requirements*

It is applicable to structures and parts of structures constructed in accordance with the materials and workmanship requirements of NZS 3109.

1.1.4 *Interpretation*

1.1.4.1 *"Shall" and "should"*

In this Standard the word "shall" indicates a requirement that is to be adopted in order to comply with the Standard. The word "should" indicates practices which are advised or recommended.

1.1.4.2 *Clause cross-references*

Cross-references to other clauses or clause subdivisions within this Standard quote the number only, for example: "... is given by 8.6.2.3 (a)"

1.1.4.3 *Commentary*

The Commentary to this Standard, NZS 3101:Part 2:2006, does not contain requirements essential for compliance with this Standard but explains, summarises technical background and suggests approaches which satisfy the intent of the Standard.

1.2 Referenced documents

The full titles of reference documents cited in this Standard are given in the "Referenced Documents" list immediately preceding the Foreword.

1.3 Design

1.3.1 *Design responsibility*

The design of a structure or the part of a structure to which this Standard is applied shall be the responsibility of the design engineer or his or her representative.

1.3.2 *Design information*

Consent documentation and the drawings or specification, or both, for concrete members and structures shall include, where relevant, the following:

(a) The reference number and date of issue of applicable design Standards used;
(b) The fire resistance ratings, if applicable;
(c) The concrete strengths;
(d) The reinforcing and prestressing steel Class and Grades used and the manufacturing method employed in the production of the reinforcing steel;
(e) The testing methods, reporting requirements and acceptance criteria for any tests of material properties, components or assemblages that are required by this Standard.
(f) The locations and details of planned construction joints;
(g) Any constraint on construction assumed in the design;
(h) The camber of any members.

1.4 Construction

1.4.1 *Construction reviewer*

All stages of construction of a structure or part of a structure to which this Standard applies shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review.

1.4.2 *Construction review*

The extent of review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

1.5 Definitions

The following terms are defined for general use in this Standard, noting that specialised definitions appear in individual sections:

ADMIXTURE. A material other than Portland cement, aggregate, or water added to concrete to modify its properties.

AGGREGATE. Inert material which is mixed with Portland cement and water to produce concrete.

ANCHORAGE. The means by which prestress force is permanently transferred to the concrete. Also, the method of ensuring that reinforcing bars and fixings acting in tension or compression are tied into a concrete member.

AXIS DISTANCE. The weighted average distance of a group of longitudinal bars or tendons from the axes of the bars to the nearest exposed surface; the weighting being conducted on the basis of bar area. A2

BEAM. An member subjected primarily to loads and forces producing flexure.

BINDER. A constituent phase of concrete, comprising a blend of cementitious materials, which on reaction bind the aggregates together into a homogenous mass.

BONDED TENDON. Prestressing tendon that is bonded to concrete either directly or through grouting.

CAPACITY DESIGN. In the capacity design of structures subjected to earthquake forces, regions of members of the primary lateral force-resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural members are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

COLUMN. An element subjected primarily to compressive axial loads.

COMPOSITE CONCRETE FLEXURAL MEMBERS. Concrete flexural members of precast and/or cast-in-place concrete elements or both, constructed in separate placements but so interconnected that all elements respond to loads as a unit.

CONCRETE. A mixture of Portland cement or any other hydraulic cement, sand, coarse aggregate and water.

CONCURRENCY. The simultaneous occurrence of actions not necessarily aligned to any principal direction of the structure, which result in actions in more than one principal direction of the structure.

CONSTRUCTION JOINT. An intentional joint in concrete work detailed to ensure monolithic behaviour at both the serviceability and ultimate limit states.

CURVATURE FRICTION. Friction resulting from bends or curves in the specified prestressing tendon profile.

DEFORMED REINFORCEMENT. Deformed reinforcing bars conforming to AS/NZS 4671.

DESIGN ENGINEER. A person who, on the basis of experience or qualifications, is competent to design structural elements of the structure under consideration to safely resist the design loads or effects likely to be imposed on the structure.

DEVELOPMENT LENGTH. The embedded length of reinforcement required to develop the design strength of the reinforcement at a critical section (see 8.6).

DIAPHRAGM. Elements transmitting in-plane lateral forces to resisting elements.

DUAL STRUCTURE. Lateral force-resisting system which consists of moment resisting frames and structural walls.

DUCTILE FRAME. A structural frame possessing ductility.

DUCTILITY. The ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake.

EFFECTIVE PRESTRESS. The stress remaining in the tendons after all calculated losses have been deducted, excluding the effects of superimposed loads and the weight of the member.

EFFECTIVE THICKNESS. The effective thickness of ribbed or hollow-core wall panels is the net cross-sectional area divided by the width.

EMBEDMENT LENGTH. The length of embedded reinforcement provided beyond a critical section.

END ANCHORAGE. Length of reinforcement, or a mechanical anchor, or a hook, or combination thereof, required to develop stress in the reinforcement; mechanical device to transmit prestressing force to concrete in a post-tensioned member.

FIRE RESISTANCE. The ability of a structure or part of it to fulfil its required functions (load-bearing and/or separating function) for a specified exposure to fire, for a specified time. Refer to prEN 1992-1-1.

FIRE RESISTANCE RATING (FRR). The term used to classify fire resistance of building elements as determined in the standard test for fire resistance, or in accordance with a specific calculation method verified by experimental data from standard fire resistance tests in accordance with AS 1530.4. It comprises three numbers giving the time in minutes for which each of the criteria for stability, integrity and insulation are satisfied.

FIRE-SEPARATING FUNCTION. The function served by the boundary elements of a fire compartment, which are required to have a fire resistance rating, in preventing a fire in that compartment from spreading to adjoining compartments.

FLAT SLAB. A two-way continuous slab supported on columns, with no beams between supporting columns.

GRAVITY LOAD DOMINATED FRAMES. A frame with full or limited ductility capacity in which the design strength of members at the ultimate limit state is governed by gravity loads rather than by the most adverse combination of gravity loads and earthquake forces.

HOLLOW-CORE SLAB OR WALL. A slab or wall having mainly a uniform thickness and containing essentially continuous voids, where the thickness of concrete between adjacent voids and the thickness of concrete between any part of a void and the nearest surface is the greater of either one-fifth the required effective thickness of the hollow-core or 25 mm. Hollow-core units have no shear reinforcement.

INSULATION. The ability of a fire-separating member, such as a wall or floor, to limit the surface temperature on one side of the member when exposed to fire on the other side.

INTEGRITY. The ability of a fire-separating member to resist the passage of flames or hot gases through the member when exposed to fire on one side.

JACKING FORCE. In prestressed concrete, the temporary force exerted by the device which introduces the tension into the tendons.

LIMIT STATE

SERVICEABILITY LIMIT STATE. The state at which a structure becomes unfit for its intended use through deformation, vibratory response, degradation or other operational inadequacy.

ULTIMATE LIMIT STATE. The state at which the design strength or ductility capacity of the structure is exceeded, when it cannot maintain equilibrium and becomes unstable.

LOADING STANDARD, REFERENCED. One of the documents referenced in C1.1.1 of the Concrete Structures Commentary which gives the range of design actions for which a structure is to be designed in order to satisfy the performance requirements of the New Zealand Building Code Clauses B1 and B2.

LOADS AND FORCES

LOAD, DEAD. The weight of all permanent components of a structure, including partitions, finishes, and permanently fixed plant and fittings.

LOAD, DESIGN. Combinations of loads and forces used in design as set out in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard for the applicable limit state. In seismic design for the ultimate limit state, the design load may be either the ultimate limit state forces or the forces resulting from the capacity design procedure depending on the case being considered.

LOAD, LIVE. Loads assumed or known to result from the occupancy or use of a structure, with values as specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

FORCE, EARTHQUAKE. Forces assumed to simulate earthquake effects as defined by AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

LOAD-BEARING FUNCTION. The ability of a structure or member to sustain specified actions under all relevant circumstances (e.g. fire – prEN 1992-1.1).

MEMBER. A physically distinguishable part of a structure such as a wall, beam, column, slab or connection.

NORMAL DENSITY CONCRETE. Concrete, excluding reinforcement with a density of between 2250 and 2350 kg/m^3.

OVERSTRENGTH. The overstrength value takes into account factors that may contribute to strength such as higher than specified strengths of the steel and concrete, steel strain hardening, confinement of concrete, and additional reinforcement placed for construction and otherwise unaccounted for in calculations.

P-DELTA EFFECT. Refers to the structural actions induced as a consequence of the axial loads being displaced laterally away from the alignment of the action.

PIER. A vertical member (usually associated with bridge structures) subjected primarily to both compressive axial loads and seismic forces.

PLAIN CONCRETE. Concrete that contains less than the minimum reinforcement required by this Standard.

PLAIN REINFORCEMENT. Reinforcing bars conforming to AS/NZS 4671 and having no significant projections other than bar identification marks.

PLASTIC HINGE (REGION). (POTENTIAL PLASTIC HINGE REGION.) Regions in a member as defined in this Standard, where significant rotations due to inelastic strains can develop under flexural actions A2

PRIMARY PLASTIC REGION. A potential plastic region identified in the ductile collapse mechanism, which is used as the basis for capacity design.

REVERSING PLASTIC HINGE. A potential plastic region which may be subjected to both negative and positive inelastic deformation in an earthquake.

SECONDARY PLASTIC REGION. A potential plastic region which may develop due to member elongation or higher mode effects in a structure.

UNIDIRECTIONAL PLASTIC HINGE. A plastic region which may be subjected to either negative or positive inelastic deformation rotation (but not both) in an earthquake.

A2 PLASTIC HINGE (REGION) LENGTH.

DUCTILE DETAILING LENGTH. ℓ_y the length in which ductile detailing is required. In this length, inelastic deformation may develop in the reinforcement or spalling may occur in the concrete.

EFFECTIVE PLASTIC HINGE (REGION) LENGTH. ℓ_p, the length used to calculate curvature (2.6.1.3.3).

EFFECTIVE PLASTIC REGION LENGTH. As for ℓ_p, but specifically identified as ℓ_{pr}, where it is used to calculate the shear strain in a diagonally reinforced coupling beam, (2.6.1.3.3).

POST-TENSIONING. A method of prestressing in which the tendons are tensioned after the concrete has hardened.

A2 POTENTIAL PLASTIC HINGE REGION. See plastic region.

PRECAST CONCRETE. A concrete element cast-in other than its final position in the structure.

PRESTRESSED CONCRETE. Concrete in which internal stresses of such magnitude and distribution have been introduced that the stresses resulting from loads are counteracted to some extent to ensure the required strength and serviceability are maintained.

PRE-TENSIONING. A method of prestressing in which the tendons are tensioned before the concrete is placed.

PRISMATIC MEMBER. A member of constant cross section along its length.

REINFORCED CONCRETE. Concrete containing steel reinforcement, and designed and detailed so that the two materials act together in resisting loads and forces.

RIBBED SLAB. A slab incorporating parallel ribs spaced at not greater than 1500 mm centre-to-centre in one or two directions.

SEGMENTAL MEMBER. A structural member made up of individual elements designed to act together as a monolithic unit under service loads.

SELF-COMPACTING CONCRETE. Concrete that flows and consolidates under its own weight without the need of vibration. SCC is characterised by high flowability, filling ability and passing ability through congested reinforcement and shall exhibit adequate static and dynamic stability.

SEPARATING FUNCTION. The ability of a separating member to prevent fire spread by passage of flames or hot gases (integrity) or ignition beyond the exposed surface (thermal insulation during the relevant fire). (Refer to prEN 1992-1-1).

SPECIAL STUDY. A procedure for justifying departure from this Standard, or for determining information not covered by this Standard, which is consistent with AS/NZS 1170.0 and its Appendices A and B.

SPECIFIED INTENDED LIFE. For a building or structure, the period of time for which the building is proposed to be used for its intended use as stated in an application for a building consent.

SPIRAL. Continuously wound reinforcement in the form of a cylindrical helix.

STABILITY. The ability of a member to maintain its structural function when deformed.

STIRRUP OR TIES. Reinforcement used to resist shear and torsion in a structural member; typically bars or wires (smooth or deformed) bent around the longitudinal reinforcement and located perpendicular to, or at an angle to longitudinal reinforcement (the term "stirrups" is usually applied to lateral reinforcement in beams and the term "ties" to those in columns). Stirrup ties or hoops may also provide confinement to compressed concrete, stability to reinforcing bars subject to compression and clamping in shear-friction mechanisms in addition to acting as shear and torsional reinforcement.

STRENGTH

STRENGTH, COMPRESSIVE OF CONCRETE. The crushing resistance of cylindrical specimens of concrete, prepared, cured and tested in accordance with the standard procedures prescribed in Sections 3, 4 and 6 of NZS 3112:Part 2. This is normally denoted by the general symbol f'_c.

STRENGTH, DESIGN. The nominal strength multiplied by the appropriate strength reduction factor.

STRENGTH, LOWER CHARACTERISTIC YIELD OF NON-PRESTRESSED REINFORCEMENT. That yield stress below which fewer than 5 % of results fall when obtained in a properly conducted test programme. Refer to AS/NZS 4671.

STRENGTH, NOMINAL. The theoretical strength of a member section, calculated using the section dimensions as detailed and the lower characteristic reinforcement strengths as defined in this Standard and the specified compressive strength of concrete.

STRENGTH, OVER. See Overstrength.

STRENGTH, PROBABLE. The theoretical strength of a member section calculated using the expected mean material strengths as defined in this Standard.

STRENGTH REDUCTION FACTOR. A factor used to multiply the nominal strength to obtain the design strength.

STRENGTH, SPECIFIED COMPRESSIVE OF CONCRETE. A singular value of strength, normally at age 28 days unless stated otherwise, denoted by the symbol f'_c, which classifies a concrete as to its strength class for purposes of design and construction. It is that level of compressive strength which meets the production standards required by Section 6 of NZS 3109.

STRENGTH, UPPER CHARACTERISTIC BREAKING STRENGTH OF NON-PRESTRESSED REINFORCEMENT. That maximum tensile strength below which greater than 95% of the results fall when obtained in a property conducted test programme.

STRUCTURAL. A term used to denote an element or elements which provide resistance to loads and forces acting on a structure.

STRUCTURAL ADEQUACY. The ability of a member to maintain its structural function when exposed to fire.

STRUCTURAL DUCTILITY FACTOR. A numerical assessment of the ability of a structure to sustain cyclic inelastic displacements.

STRUCTURAL LIGHTWEIGHT CONCRETE. A concrete containing lightweight aggregate and having a unit weight not exceeding 1850 kg/m^3. In this Standard, a lightweight concrete without natural sand is termed "all-lightweight concrete", and lightweight concrete in which all of the fine aggregate consists of normal density sand is termed "sand-lightweight concrete" .

STRUCTURAL PERFORMANCE FACTOR. A factor which is used in the derivation of design earthquake forces in accordance with AS/NZS 1170 and NZS 1170.5 or other referenced loading standard and 2.6.2.2 of this Standard.

SUPPLEMENTARY CROSS TIES. Additional ties placed around longitudinal bars supplementing the functions of stirrups or ties.

TENDON. A steel element such as wire, cable, bar, rod, or strand which when tensioned imparts a prestress to a concrete member.

TIES. See Stirrups.

TRANSFER. Act of transferring stress in prestressing tendons from jacks or pre-tensioning bed to a concrete member.

UNBONDED TENDONS. Tendons which are not bonded to the concrete either directly or through grouting. They are usually wrapped in a protective and lubricating coating to ensure that this condition is obtained.

WALL. Means a structural wall, a vertical thin member, usually planar, which because of its position, strength, shape, and stiffness, contributes to the rigidity and strength of a structure.

WOBBLE FRICTION. In prestressed concrete, the friction caused by the unintended deviation of the prestressing sheath or duct from its specified profile.

2 DESIGN PROCEDURES, LOADS AND ACTIONS

2.1 Notation

A moment ratio for coupled walls

A_r aspect ratio of wall = h_w/L_w

A_{sk} area of a bar used as skin reinforcement on the side of a beam, wall or column, mm^2

c distance from extreme compression fibre to neutral axis, mm

c_c clear cover between the reinforcement and the surface of the concrete, mm

c_m cover distance measured from the centre of the reinforcing bar, mm

d effective depth, distance from extreme compression fibre to centroid of tension reinforcement, mm

E_s modulus of elasticity of reinforcing steel, MPa

F_{ph} inertia force used in design of a part, N

f'_c specified compressive strength of concrete, MPa

f_s steel stress at the serviceability limit state, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

G dead load, N, kPa or N/mm

g_s distance from centre of reinforcing bar to a point on surface of concrete where crack width is being assessed, mm

h overall depth of the member measured at right angles to the axis of bending, mm | A2

h_b overall beam depth, mm

h_c overall depth of column in the direction of the horizontal shear force, mm

h_w height of wall, mm

k ratio of depth of neutral axis to effective depth, d, of member based on elastic theory for members cracked in flexure

k_1 factor for determining minimum slab thickness, see 2.4.3

ℓ_p length used to calculate curvature in a plastic hinge (region), (2.6.1.3.3), mm | A2

ℓ_{pr} length used to calculate shear strain in a diagonally reinforced coupling beam, (2.6.1.3.3), mm

L effective span length of beam, girder or one-way slab, as defined in 6.3.3(a); clear projection of cantilever, mm

L' centre-to-centre distance of coupled walls, mm

L_n length of clear span in long direction of two-way construction, measured face-to-face of columns in slabs without beams and face-to-face of beams or other supports in other cases, mm

L_{nd} length of region of beam containing diagonal reinforcement, mm | A2

L_s shortest span length of bridge deck slab, mm

L_w horizontal length of wall in-plane of loading, mm

M^* design moment action for ULS, N mm

M_n nominal flexural strength, N mm

M_s maximum bending moment calculated for serviceability limit state load combination with long-term live load, N mm

M^*_o overstrength bending moment, N mm

M_{ow} total over turning moment at base of a structure comprising structural walls due to lateral design earthquake forces, N mm

N^*_o axial load that acts simultaneously with overstrength bending moment, N

p proportion of flexural tension reinforcement | A2

Q live load, N, kPa, or N/mm

S_n nominal strength at the ultimate limit state for the relevant action of moment, axial load, shear or torsion, N or N mm

S_p structural performance factor

S^* design action at the ultimate limit state, N or N mm

s centre-to-centre spacing of reinforcing bars, mm

t thickness of member, mm

T_w axial load at the base of each coupled structural wall induced by design earthquake forces, N

V^* design shear action in ULS, N
w design crack width due to flexure, mm
y distance from the extreme compression fibre to the fibre being considered, mm
Z_t section modulus related to extreme tension fibre calculated from gross section properties at the section sustain the maximum bending moment, mm^3
α ratio of the flexural stiffness of beam to the flexural stiffness of a width of slab bounded laterally by the centrelines of adjacent panels, if any, on each side of the beam, see Table 2.2
α_{fy} a factor used in assessing permissible curvature limits in plastic regions
α_m average value of α for all beams on the edges of a panel
β ratio of clear spans in long to short direction of two-way slabs
β' ratio used to find strain in section in 2.4.4.6
β_a factor to determine ductility factor for walls, see Table 2.5
ε_y yield strain of reinforcement
μ structural ductility factor
ϕ strength reduction factor as defined in 2.3.2.2 and 2.6.3.2
A2 | ϕ_{max} limiting curvature, radians/mm
$\phi_{o,fy}$ overstrength factor depending on reinforcement grade, see 2.6.5.6
A2 | ϕ_y curvature at first yield, radians/mm
ω dynamic magnification factor
ψ_s short-term live load factor (see AS/NZS 1170)

2.2 Design requirements

2.2.1 *Design considerations*

The structure and its component members shall be designed to satisfy the requirements of this Standard for stiffness, strength, stability, ductility, robustness, durability and fire resistance.

2.2.2 *Design for strength and serviceability*

Concrete structures shall be designed for ultimate strength and serviceability limit states in accordance with the general principles and procedures for design as set out in AS/NZS 1170:Part 0 or other referenced loading standard and the specific requirements of 2.3 to 2.6.

2.2.3 *Design for robustness, durability and fire resistance*

Concrete structures shall be designed to be:

(a) Robust in accordance with the procedures and criteria given in Part 0 of AS/NZS 1170 or other referenced loading standard;
(b) Durable in accordance with the procedures and criteria given in Section 3; and
(c) Fire resistant in accordance with the procedures and criteria given in Section 4.

2.2.4 *Material properties*

The properties of materials used in the design shall be in accordance with Section 5.

2.3 Design for strength and stability at the ultimate limit state

2.3.1 *General*

The structure and its component members shall be designed for the ultimate limit state by providing stiffness, strength and ductility and ensuring stability, as appropriate, in accordance with the relevant requirements of 2.3.2 to 2.3.3.

2.3.2 *Design for strength*

2.3.2.1 *General*

The design shall consider and take into account the construction sequence, the influence of the schedule for stripping of formwork and the method of back-propping on the loading of the structure during construction and their effect on the strength and deflection of the structure. The structural effects of differential settlement of foundation elements and lateral movement of the ground shall be considered where appropriate, and provided for in accordance with this Standard.

Structures and structural members shall be designed for strength as follows:

(a) The loads and forces giving rise to the ultimate limit state design action, S^*, shall be determined from the governing ultimate limit state combinations specified in AS/NZS 1170 or other referenced loading standard;

(b) The design strength of a member or cross section at the ultimate limit state shall be taken as the nominal strength, S_n, for the relevant action calculated in accordance with the requirements and assumptions of this Standard, multiplied by the applicable strength reduction factor, ϕ, specified in 2.3.2.2;

(c) Each member shall be proportioned so that the design strength is equal to or greater than the design action, in accordance with the following relationship:

$$S^* \le \phi S_n \quad \text{(Eq. 2–1)}$$

where S is replaced in Equation 2–1 by the actions of moment, axial force, shear or torsion as appropriate.

2.3.2.2 *Strength reduction factors, ultimate limit state*

The strength reduction factor, ϕ, shall be as follows:

(a)	For actions which have been derived from overstrengths of elements in accordance with the principles of capacity design (see 2.6.5)	1.00
(b)	Anchorage and strength development of reinforcement	1.00
(c)	Flexure with or without axial tension or compression	0.85
(d)	Shear and torsion	0.75
(e)	Bearing on unconfined concrete	0.65
(f)	Bearing on confined concrete (See 16.3.3)	0.85
(g)	Tension in plain concrete	0.60
(h)	Strut and tie models	0.75
(i)	Corbels	0.75
(j)	For design under fire exposure	1.00

2.3.3 *Design for stability*

For ultimate limit state load combinations, the structure as a whole and its component members shall be designed to prevent instability in accordance with AS/NZS 1170 or other referenced loading standard.

2.4 Design for serviceability

2.4.1 *General*

2.4.1.1 *Deflection, cracking and vibration limits*

The structure and its component members shall be designed for the serviceability limit state by limiting deflection, cracking and vibration, where appropriate, in accordance with the relevant requirements of 2.4.2 to 2.4.4.

2.4.1.2 *Vibration*

Appropriate measures shall be taken to evaluate and limit where necessary the effects of potential vibration from wind forces, machinery and vehicular, pedestrian or traffic movements on the structure, to prevent discomfort to occupants or damage to contents.

2.4.1.3 *Seismic actions*

Where seismic actions are included in a load combination the structure shall be proportioned to meet the requirements of 2.6.3.

2.4.1.4 *Strength reduction factor*

Where it is necessary to check or design for the strength associated with wind or seismic serviceability load combinations a strength reduction factor not exceeding 1.1 shall be used.

2.4.2 *Deflection*

2.4.2.1 *Structures other than bridges*

Deflection in concrete structures and members shall either be determined in accordance with 6.8 or the minimum thickness provisions of 2.4.3 shall be applied.

The deflections computed in accordance with 6.8 shall, where required, meet the limits given by AS/NZS 1170, or for earthquake loading NZS 1170.5, or another referenced loading standard for the relevant serviceability limit state criteria.

2.4.2.2 *Bridges*

The design of bridge girders shall mitigate the deflection due to the combination of permanent loads, shrinkage, prestress and creep over the long-term to ensure appropriate ride quality and drainage of the bridge deck.

2.4.3 *Minimum thickness for buildings*

The minimum thickness of non-prestressed beams and slabs subjected to gravity load combinations may be determined by calculation, as specified in 6.8 or by satisfying the minimum span to depth ratios and other requirements given in (a), (b), or (c) below, as appropriate.

(a) *One-way spans*

The limiting span to depth ratios shall only be used for determining the minimum thickness of non-prestressed beams or slabs where the condition in Equation 2–2 is satisfied. Where this condition is not satisfied deflection calculations shall be made as specified in 6.8. In Equation 2–2, M_s is the maximum bending moment in the serviceability limit state due to dead load and long-term live load calculated assuming uniform elastic properties, k_1 is a factor given in the Table 2.1 and Z_t is the section modulus for the extreme tension fibre calculated from the gross section.

$$M_s < k_1 \sqrt{f_c'}\ Z_t \quad \text{(Eq. 2–2)}$$

Table 2.1 – Minimum thickness of non-prestressed beams or one-way slabs

f_y (MPa)	Member	Minimum thickness, h and value of k_1							
		Members not supporting or attached to partitions or other construction likely to be damaged by large deflections							
		Simply supported		One end continuous		Both ends continuous		Cantilever	
		h	k_1	h	k_1	h	k_1	h	k_1
300	Solid one-way slabs	$\frac{L}{25}$	1.0	$\frac{L}{30}$	1.1	$\frac{L}{35}$	1.2	$\frac{L}{13}$	1.0
	Beams or ribbed one-way slabs	$\frac{L}{20}$	1.0	$\frac{L}{23}$	1.0	$\frac{L}{26}$	1.0	$\frac{L}{10}$	1.0
500	Solid one-way slabs	$\frac{L}{18}$	1.0	$\frac{L}{20}$	1.1	$\frac{L}{25}$	1.2	$\frac{L}{9}$	1.0
	Beams or ribbed one-way slabs	$\frac{L}{14}$	1.0	$\frac{L}{16}$	1.0	$\frac{L}{19}$	1.0	$\frac{L}{7}$	1.0

NOTE –
The values given shall be used directly for members with normal density concrete ($\rho \approx 2400$ kg/m^3). For lightweight concrete having a density in the range of 1450-1850 kg/m^3, the values shall be multiplied by $(1.65 - 0.0003\rho)$ where ρ is the density in kg/m^3.

(b) *Two-way construction (non-prestressed) for buildings*

For non-prestressed two-way slabs for buildings the minimum thickness of slabs without interior beams spanning between the supports shall be in accordance with the provisions of Table 2.2 and shall be equal to or greater than the following values dependant on the provision of drop panels that conform with 12.5.6.1:

(i) Slabs without drop panels ..... 120 mm
(ii) Slabs with drop panels ..... 100 mm

Table 2.2 – Minimum thickness of slabs without interior beams

f_y (MPa)	Without drop panels [1]			With drop panels [1]		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams [2]		Without edge beams	With edge beams [2]	
300	$\frac{L_n}{33}$	$\frac{L_n}{36}$	$\frac{L_n}{36}$	$\frac{L_n}{36}$	$\frac{L_n}{40}$	$\frac{L_n}{40}$
500	$\frac{L_n}{28}$	$\frac{L_n}{31}$	$\frac{L_n}{31}$	$\frac{L_n}{31}$	$\frac{L_n}{34}$	$\frac{L_n}{34}$

NOTE –
(1) Drop panel is defined in 12.5.6.1.
(2) Slabs with beams between columns along exterior edges. The value of α for the edge beam shall be equal to or greater than 0.8.

(c) *For slabs supported by beams on all four sides, the minimum thickness shall depend on the value of* α_m, *as given below*:
(i) For α_m equal to or less than 0.2 the limits given in Table 2.2 shall apply;
(ii) For α_m between the limits of 0.2 and 2.0 the thickness shall be equal to or greater than:

$$h = \frac{L_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 5\beta(\alpha_m - 0.2)} \geq 120 \text{ mm} \quad \text{.....(Eq. 2–3)}$$

where α_m is the average value of α for all the beams and α is defined in 2.1.
(iii) For α_m greater than 2.0 the thickness shall be equal to or greater than:

$$h = \frac{L_n\left(0.8 + \frac{f_y}{1500}\right)}{36 + 9\beta} \geq 120 \text{ mm} \quad \text{.....(Eq. 2–4)}$$

(iv) For slabs without beams, but with drop panels extending in each direction from the centreline of support a distance equal to or greater than one-sixth the span length in that direction measured centre-to-centre of the supports, and a projection below the slab of at least one quarter of the slab thickness beyond the drop, the thickness required by Equations 2–3, or 2–4 may be reduced by 10 %.
(v) At discontinuous edges one of the following conditions shall be satisfied:
(A) An edge beam with a stiffness ratio, α, equal to or greater than 0.8 shall be provided;
(B) The minimum thickness of the slab shall be equal to or greater than the value given by Equation 2–3;
(C) The minimum slab thickness given by Equation 2–4 shall be increased by at least 10 % for the panel with the discontinuous edge.

(d) *Composite precast and in situ concrete construction for buildings*
If the thickness of non-prestressed composite members meets the requirements of Table 2.1, deflection need not be calculated except as required by 6.8.5 for shored construction.

(e) *Bridge structure members*
The minimum thickness stipulated in Table 2.3 shall apply to flexural members of bridge structures unless calculation of deflection and design for the effects of traffic-induced vibration calculated in

accordance with engineering principles indicates that a lesser thickness may be used without adverse effect.

Table 2.3 – Minimum thickness of prismatic flexural members of bridge structures

Superstructure type	Minimum thickness	
	Simple spans	Continuous spans
Bridge deck slabs	$1.2\left(100+\frac{L_s}{30}\right)$	$100+\frac{L_s}{30}$
T-girders	$0.070L$	$0.065L$
Box-girders	$0.060L$	$0.055L$

A2 NOTES –
(1) For non-prismatic members the values given may be adjusted to account for change in relative stiffness of positive and negative moment sections.
(2) Slab span length, L_s, shall be determined in accordance with 12.8.4.

2.4.4 *Crack control*

2.4.4.1 *Cracking due to flexure and axial load in reinforced concrete members in buildings*

Crack widths for serviceability load combinations involving any combination of gravity loads and lateral forces excluding earthquake actions and wind actions shall be controlled by satisfying one of the following sets of criteria:

(a) Crack control measures need not be considered where the maximum longitudinal tensile stress calculated from gross section properties is equal to or less than $0.4\sqrt{f_c'}$ (MPa);

(b) The reinforcement shall be distributed and the maximum stress levels limited so that the requirements of 2.4.4.3, 2.4.4.4, 2.4.4.5 and 2.4.4.7 are satisfied;

(c) The reinforcement shall be distributed in the tension zone so that the maximum crack width calculated from 2.4.4.6 does not exceed an acceptable limit.

2.4.4.2 *Bridges*

Calculated crack widths in surfaces of bridge superstructures and exposed surfaces of bridge substructures shall not exceed those specified in the Transit New Zealand Bridge Manual.

2.4.4.3 *Cracking due to flexure and axial load in prestressed concrete members*

For crack control in prestressed members see 19.3.3.

2.4.4.4 *Spacing of reinforcement for crack control on the extreme tension face*

The spacing of deformed reinforcement, s, crossing a potential crack and located next to the tension face of a member, shall be smaller than the values given by either:

$$s=\frac{90000}{f_s}-2.5c_c \quad \text{.....(Eq. 2–5)}$$

or

$$s=\frac{70000}{f_s} \quad \text{.....(Eq. 2–6)}$$

where f_s is the stress in the reinforcement at the serviceability limit state and c_c is the clear cover between the reinforcement and the surface of the concrete.

2.4.4.5 *Crack control on the sides of members subjected to tension*

Structural members subjected to tension due to bending, or bending and axial tension in the serviceability limit state, which have an overall depth, h, of 1.0 m or more, shall have longitudinal reinforcement uniformly distributed along both sides of the member for a distance of $h/2$ from the extreme tension fibre of

the member. This longitudinal skin reinforcement shall be placed parallel to each face, with spacing, *s*, which is less than the smallest of:

(a) 300 mm

(b) *h*/6

(c) 3*t*, where t is the thickness of the wall or web of the member

(d) $\frac{1000\,A_{sk}}{h-750}$

where A_{sk} is the area of a bar used as skin reinforcement. The skin reinforcement may be included in calculations to determine the flexural strength of the member, and the total area of skin reinforcement on both sides need not exceed half of the required flexural tension reinforcement.

2.4.4.6 *Assessment of surface crack widths*

Where limitations are placed upon the desirable crack width, the design surface crack width, *w*, for members reinforced with deformed bars may be assessed from the equation:

$$w = 2.0\,\beta'\,\frac{f_s}{E_s}\,g_s \qquad \text{(Eq. 2-7)}$$

where f_s/E_s is the strain at the level of the reinforcement, determined by standard flexural theory for transformed elastic sections,

β', is a coefficient, given by:

$$\beta' = \frac{y - kd}{d - kd} \qquad \text{(Eq. 2–8)}$$

where

kd is the depth of the neutral axis, and

g_s is the distance from the centre of the nearest reinforcing bar to the surface of the concrete at the point where the crack width is being calculated, and

y is the distance from the extreme compression fibre to the fibre being considered

For the case where a crack width is being calculated between two bars the critical value of g_s is given by:

$$g_s = \sqrt{(s/2)^2 + c_m^2} \qquad \text{(Eq. 2–9)}$$

where

c_m is the cover distance measured from the centre of the bar to the surface of the concrete, and

s is the centre-to-centre spacing of the bars

2.4.4.7 *Crack control in flanges of beams*

Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over the effective flange overhang width defined in 9.3.1.3 to control crack widths. Consideration should also be given to adding reinforcement outside this width to control cracking. A2

2.4.4.8 *Control of thermal and shrinkage cracking*

Cracking of concrete due to differential temperature, heat of hydration or shrinkage of concrete shall be determined from first principles, where these actions may lead to a loss of serviceability of the structure.

Potential cracking due to plastic shrinkage shall be controlled by specification.

2.5 Other design requirements

2.5.1 *General*

Requirements such as those for fatigue, removal or loss of support, together with other performance requirements shall be considered in the design of the structure in accordance with established engineering principles.

2.5.2 *Fatigue (serviceability limit state)*

2.5.2.1 *General*

The effects of fatigue shall be considered where the imposed loads and forces on a structure are repetitive in nature.

2.5.2.2 *Permissible stress range*

A2 At sections where frequent stress fluctuations occur, the stress range in reinforcing bars, excluding stirrups and ties, caused by the repetitive loading at the serviceability limit state, shall be equal to or less than the appropriate limit given in either (a) or (b) below:

(a) The stress range shall be equal to or less than the value given in the Table below, where D is the diameter of the bend measured to the inside of the bar and d_b is the diameter of the bar.

Stress range, MPa	150	135	120	90	50
D/d_b	>25	20	15	10	5

Interpolation may be used for intermediate values of D/d_b.

(b) Appropriate values are found from a special study in which the influence of the following factors is considered:
   (i) The shape of deformations and bar marks;
   (ii) The composition and diameter of the reinforcement;
   (iii) The method of manufacture;
   (iv) The diameter of bends in the reinforcement;
   (v) The influence of embedment of the bar in cracked concrete;
   (vi) The histogram of stress variation over the expected life of the structure.

2.5.2.3 *Highway bridge fatigue loads*

For highway bridges, the vehicle loading specified by the Transit New Zealand Bridge Manual shall be used as a basis for assessing the fatigue stress range.

2.6 Additional design requirements for earthquake effects

2.6.1 *General*

2.6.1.1 *Deformation capacity*

In addition to the requirements of 2.3.2 for strength, the structure and its component parts shall be designed to have adequate ductility at the ultimate limit state for load combinations including earthquake actions.

2.6.1.2 *Classification of structures*

Structures subjected to earthquake forces shall be classified for design purposes as brittle structures, nominally ductile structures, structures of limited ductility or ductile structures, as specified below:

(a) Brittle concrete structures shall be those structures that contain primary seismic resisting members, which do not satisfy the requirements for minimum longitudinal and shear reinforcement specified in

this Standard, or rely on the tensile strength of concrete for stability. Brittle structures are not considered in this Standard.

(b) Nominally ductile structures are those that are designed using a structural ductility factor of 1.25 or less.

(c) Structures of limited ductility are a sub-set of ductile structures, which are designed for a limited overall level of ductility. The structural ductility factor shall not exceed 3.0.

(d) Ductile structures are those structures designed for a high level of ductility. The structural ductility factor shall not exceed 6.0.

2.6.1.3 *Classification of potential plastic regions*

2.6.1.3.1 *Classification nomenclature*

Potential plastic regions shall be classified for the purpose of defining the required detailing as:

(a) Nominally ductile plastic region, NDPR;

(b) Limited ductile plastic region, LDPR;

(c) Ductile plastic region, DPR.

2.6.1.3.2 *Material strain limits in plastic regions* A2

The classification depends on the level of deformation that each potential plastic region can safely sustain at the ultimate limit state. For plastic hinge regions the material strain is taken as curvature, while for diagonally reinforced coupling beams the material strain is taken as the average shear strain over the length of the diagonally reinforced portion of the beam, L_{nd}. The strain limits for different classifications of potential plastic regions are given in 2.6.1.3.4 These limits shall not be exceeded except where a special study (see AS/NZS 1170.0 and NZS 1170.5) shows that higher strain levels can be sustained with a high level of confidence.

To determine the material strain in a critical plastic hinge region, find the plastic hinge rotation or the shear displacement in a diagonally reinforced coupling beam. For a plastic hinge the rotation shall be divided by the appropriate effective plastic hinge length, ℓ_p, to give a nominal curvature. For shear deformation in a diagonally reinforced coupling beam the shear displacement shall be divided by the effective plastic region length, ℓ_{pr}, to give a nominal shear strain. The appropriate values of ℓ_p and, ℓ_{pr} are defined in 2.6.1.3.3.

Where the analysis of the structure has been based on an equivalent static or modal analysis, the rotation in a plastic hinge (region) or shear displacement in a diagonally reinforced coupling beam, shall be found by adding the elastic deformation given in (a) to the inelastic deformation given in (b) below:

(a) The elastic rotation of a plastic hinge shall be taken as the elastic curvature times the effective plastic hinge length, ℓ_p. The elastic curvature, ϕ_y, is given by $\phi_y = \dfrac{2f_y}{E_s h}$ but the value of f_y in this expression is not taken as greater than 425 MPa, or a lower value defined in 2.6.1.3.4(b), for cases where shear strength determines the strength of the member. The elastic shear displacement in a diagonally reinforced coupling beam shall be taken as the deformation sustained when the length of reinforcement contained within the beam length, L_{nd}, plus additional lengths of one-quarter of the development length of the bars at each end of the beam, sustains its design yield stress;

(b) The corresponding inelastic deformation shall be calculated from (i), (ii) or (iii) below as appropriate:

  (i) For reversing plastic regions the critical inelastic rotation or shear deformation shall be calculated from the deflected shape profiles defined in NZS 1170.5 clauses 7.2 and 7.3 amplified by the modification factor for inter-storey drift, but excluding the deformation associated with elastic response;

  (ii) For unidirectional plastic regions, the inelastic rotation, either adjacent to the column faces or in the span of the beam, shall be calculated as for reversing plastic hinges adjacent to the column face but multiplied by:

A2

$1.63(\mu - 1)$ for $\mu \leq 2.0$

$1.63\sqrt{(\mu - 1)}$ for $2.0 < \mu < 6$

where μ is the structural ductility factor

(iii) The corresponding values from another appropriate referenced loading standard.

Where time history analysis has been used, in which inelastic deformation characteristics of members are modelled and P-delta actions are included in the analysis, the required inelastic deformation shall be taken as the critical rotation or shear deformation sustained in the plastic region being considered for the suite of analyses using earthquake ground motions which meet the requirements of NZS 1170.5, clauses 6.4 and 7.3, or other appropriate referenced loading standard.

2.6.1.3.3 *Effective plastic region lengths*

The effective plastic hinge length, ℓ_p, used to calculate curvature, and the effective plastic region length, ℓ_{pr}, used to calculate an average shear strain in a diagonally reinforced coupling beam, shall be found from either a special study (see AS/NZS 1170.0 and NZS 1170.5), or taken from the appropriate value given below:

(a) For reversing plastic hinge regions in beams, columns and out of plane loading in walls, the effective plastic hinge length, ℓ_p, shall be taken as the smaller of:
   (i) (A) 0.5 h_b for a beam;
   (B) 0.5 h_c for a column;
   (C) t for a wall where $t \leq 200$ mm and 0.5 t where $t \geq 500$ mm. Linear interpolation may be used between these limits;
   (ii) The larger of 0.25 M^*/ V^*,or:
   (A) 0.25 h_b for a beam;
   (B) 0.25 h_c for a column;
   (C) Half the value given in (i)(C) for a wall;

(b) For reversing plastic hinge regions in walls for in-plane loading, the effective plastic length, ℓ_p, shall be taken as the smaller of:
   (i) Half the length of the wall, 0.5 L_{wi}
   (ii) 0.15 M^*/ V^*.

(c) For unidirectional plastic hinges, where inelastic rotation can develop on both sides of the critical section, ℓ_p, may be taken as twice the corresponding value found in (a) or (b) above.

(d) For diagonally reinforced coupling beams complying with 9.4.4.1.7, the effective plastic region length for shear, ℓ_{pr}, shall be taken to equal the length of longitudinal projection of diagonal reinforcement, but not exceeding the clear span of the beam (L_{nd}).

The moment to shear ratio, M^*/ V^*, shall be the value acting at the critical section of the plastic region calculated from the critical ultimate limit state load combination with seismic actions. Where the modal response spectrum method is used the M^*/ V^* ratio may be replaced by the M / V ratio from a corresponding equivalent static or first mode analysis. Allowance may be made for the change in actions associated with moment redistribution, but no allowance should be made for dynamic magnification effects.

2.6.1.3.4 *Material strain limits*

A2

Curvatures in plastic regions and shear strains in diagonally reinforced coupling beams shall not exceed the appropriate limits given in (a) to (d) below:

(a) For nominally ductile plastic regions where the strength is limited by flexure, the limiting curvature depends on the form of plastic region (unidirectional or reversing) and the details of reinforcement in the member:
   (i) For unidirectional plastic regions, the limiting curvature shall be equal to or less than the smaller of $0.004/c$ or $0.02/(d - c)$,
   (ii) For reversing plastic regions in all walls for both in plane and out of plane bending, and in beams and columns where the longitudinal reinforcement on the compression side or the member is

restrained against buckling by ties, which satisfy the spacing limits in 9.3.9.6.2 and 9.3.9.6.3 and longitudinal reinforcement, which satisfies 9.3.8.2.1 and 9.3.8.2.2 regardless of whether the bars are required to act in compression or not, the limiting curvature shall be equal to or less than the smaller of $0.0024/c$ or $0.012/(d - c)$. A2

(iii) For reversing plastic regions, where either the minimum area or the requirements for the restraint of reinforcement on the compression side of the member detailed in (ii) are not satisfied, the limiting curvature shall be equal to or less than:

$$\frac{2f_y}{E_s h}$$

where f_y shall not be taken greater than 425 MPa and h is the overall depth of the member for bending.

(b) For nominally ductile plastic regions where the nominal strength of the member is limited by shear, the limiting curvatures shall be equal to or less than:

$$\frac{2f_s}{E_s h}$$

where f_s is the maximum stress in the flexural tension reinforcement when the member is sustaining 85 % of its nominal shear strength.

(c) Limited ductile and ductile plastic regions

The limiting curvature, ϕ_{max}, in plastic regions in beams, columns and walls, shall be equal to or less than the appropriate value given by:

$\phi_{max} = K_d \phi_y$

Where ϕ_y is given by:

$$\phi_y = \frac{2f_y}{E_s h}$$

Where f_y in the expression for ϕ_y is not taken greater than 425 MPa and K_d is a factor given in Table 2.4(a) and (b).

(d) For diagonally reinforced coupling beams complying with 9.4.4.1.7, the shear strain shall be equal to or less than 0.035 radians. No flexural deformation shall be included in determining the ultimate deformation limit for the member.

Table 2.4 (a) – K_d factor for limiting curvatures in flexural plastic regions in beams and columns

Classification of plastic region	Type of plastic hinge	Limiting curvature K_d
Limited ductile plastic region	Unidirectional	22
	Reversing	11
Ductile region	Unidirectional	38
	Reversing	19

Table 2.4 (b) – K_d factor for limiting curvatures in walls

Wall classification	Limited ductile plastic region	Ductile plastic region
Singly reinforced	5	n/a
Doubly reinforced	6	14
Doubly reinforced with confined boundary elements (see NOTE)	9	16

NOTE –
A wall may be assumed to have confined boundary elements where the boundary elements satisfy the dimensional requirements of 11.4.2 and the wall including the boundary elements satisfy the requirements of 11.4.6.

2.6.1.4 *Stiffness of members for seismic analysis*

Assessment of structural deflections involving seismic forces shall make due allowance for the anticipated levels of concrete cracking associated with the strain levels sustained by the reinforcement and with the quantity of longitudinal reinforcement.

(a) For the serviceability limit state and for members, which are not expected to sustain inelastic deformation in the ultimate limit state, allowance for flexural cracking shall be consistent with the maximum expected strain levels in the members.

(b) For the ultimate limit state, where elastic-based methods of analysis are used (equivalent static, modal response spectrum or elastic time history), the stiffness of members that are expected to sustain plastic deformation in a design level earthquake shall correspond to the stiffness under cyclic loading conditions to first yield of the member. For other members the stiffness should be consistent with the expected maximum stress level induced in the member when adjacent potential plastic regions are sustaining their nominal strengths. Any potential increase in actions above this level due to overstrength of potential plastic regions or due to dynamic magnification effects, should be ignored for the purpose of assessing stiffness.

Assessment of structural deflections for the ultimate limit state involving seismic forces shall make due allowance for the anticipated levels of post-elastic effects and P-delta actions, as specified in NZS 1170.5.

2.6.2 *Seismic design actions*

2.6.2.1 *General*

In the derivation of seismic actions for the serviceability and ultimate limit states, the design actions specified by NZS 1170.5, or other referenced loading standard, shall be found using:

(a) A structural performance factor which is equal to or greater than the appropriate value for the limit state being considered, as given in 2.6.2.2;

(b) A structural ductility factor which is equal to or less than the maximum appropriate value given 2.6.2.3;

(c) The dynamic characteristics of the structure;

A2 (d) The design response spectrum, return period factor and seismic zone factor, given in NZS 1170.5 or other referenced loading standard.

2.6.2.2 *Structural performance factor*

2.6.2.2.1 *S_p values*

The structural performance factor, S_p, shall be taken as equal to or greater than:

(b) For the serviceability limit state ..... $S_p = 0.7$

(c) For the ultimate limit state:

(i) For nominally ductile structures ..... $S_p = 0.9$

(ii) For structures with a structural ductility factor of 3 or more ..... $S_p = 0.7$

Interpolation may be used between these limits.

2.6.2.2.2 *Lower S_p may be used when detailing requirements met*

For nominally ductile structures and structures with a structural ductility factor of less than 3, an S_p factor of 0.7 may be used for determining the seismic actions provided all the potential plastic regions are
A2 detailed as required for limited ductile plastic regions (LDPR) or ductile plastic regions (DPR), unless a particular clause gives the value of S_p to be used in determining the design actions associated with that clause.

2.6.2.3 *Structural ductility factor*

2.6.2.3.1 *SLS*

A2 The structural ductility factor, μ, shall be unity for the SLS1 serviceability limit state and equal to or less than 2 for the SLS2 serviceability limit state.

2.6.2.3.2 *ULS*

For the ultimate limit state two factors need to be considered in determining the structural ductility factor:

(a) The selected value shall not exceed the appropriate value given in Table 2.5;

(b) The value of the structural ductility factor shall be such that the maximum permissible material strain limit is not exceeded in the critical plastic region.

Table 2.5 – Maximum available structural ductility factor, μ, to be assumed for the ultimate limit state

Type of structure	Reinforced concrete	Prestressed concrete with bonded non-prestressed reinforcement
1. Nominally ductile structures	1.25	1.25
2. Structures of limited ductility		
(a) Moment resisting frame	3	3
(b) Walls	3	3
(c) Cantilever face loaded walls (single storey only)	2	2
3. Ductile structures		
(a) Moment resisting frame	6	5
(b) Wall		
(i) Two or more cantilevered	$\frac{5}{\beta_a}$	As for reinforced concrete
(ii) Two or more coupled	$\frac{5}{\beta_a} \le \frac{3A+4}{\beta_a} \le \frac{6}{\beta_a}$	As for reinforced concrete
(iii) Single cantilever	$\frac{4}{\beta_a}$	As for reinforced concrete

NOTE –

(1) The ductility factor is a measure of the anticipated overall structural ductility demand which is a function of the appropriate magnitude of earthquake design forces.

(2) In the above table

$1.0 < \beta_a = 2.5 - 0.5A_r < 2.0$

and

$$\frac{1}{3} \le A = \frac{T_w L'}{M_{ow}} \le \frac{2}{3}$$

2.6.3 *Serviceability limit state*

2.6.3.1 *General*

The structure shall be proportioned to meet the serviceability requirements of NZS 1170.5 or other referenced loading standard. An analysis to determine the seismic induced deformations and inter-storey drifts for the serviceability limit state shall be made by using either method (a), or method (b), as detailed below.

(a) An elastic analysis may be used to determine the deformations sustained provided one of the two following conditions is satisfied:

(i) The structure is designed as a nominally ductile structure or a structure of limited ductility;

(ii) The members are proportioned so that the design strength exceeds the design actions for the critical serviceability seismic load combinations.

(b) Allowance is made, using engineering principles, for the influence of inelastic deformation of members under the action of load combinations including seismic forces and gravity loads. The analysis shall include, where appropriate, calculation of increased deflection of members due to shake down effects and determination of residual inter-storey drifts and crack widths.

2.6.3.2 *Strength reduction factor*

Strength checks for seismic load combinations shall be made using standard ultimate strength theory with a strength reduction factor, ϕ, which is equal to or less than 1.1.

2.6.4 *Ultimate limit state*

(e) For all structural classifications (2.6.1.2) the structure shall be proportioned such that:
  (i) The design strengths shall be equal to or exceed the design actions;
  (ii) The inter-storey drift limits and P-Delta stability coefficients given in NZS 1170.5 or other referenced loading standard, shall not be exceeded;
  (iii) The maximum permissible lateral displacement of the structure at site boundaries, as specified in NZS 1170.5, or other referenced loading standard, shall not be exceeded.

(f) Structures of nominal ductility, limited ductility and ductile structures, shall be proportioned to ensure that when the maximum lateral displacements for the ultimate limit state act on the structure, the material strains sustained in critical potential plastic regions do not exceed the maximum permissible values for the level of detailing that is used.

2.6.5 *Capacity design*

2.6.5.1 *General*

All ductile structures and structures of limited ductility shall be proportioned to meet the requirements of
A2 | capacity design following the procedure outlined in NZS 1170.5 and 2.6.5.

2.6.5.2 *Identification of ductile mechanism*

In capacity design it is assumed that the structure is displaced laterally so that primary plastic regions form to give a ductile failure mechanism. A permissible failure mechanism shall be selected and potential primary plastic regions identified, see 2.6.7. The required seismic lateral forces to develop the primary plastic regions shall be assumed to act simultaneously with dead, and where appropriate long-term live load (for example G & ψ_ℓQ in AS/NZS 1170).

2.6.5.3 *Detailing of potential plastic regions*

The material strains in the critical potential plastic regions shall be assessed from the deformed shape in the ultimate limit state, as defined in NZS 1170.5 or other referenced loading standard, and from the appropriate length of the plastic regions as identified in 2.6.1.3. The magnitude of the material strain shall be used to identify the appropriate level of detailing.

2.6.5.4 *Overstrength actions*

The overstrength actions shall be determined for each potential primary plastic region on the basis of:

(a) The detailing used in the region;

(b) The critical load combinations which may occur in each region;

(c) The likely maximum material strengths in each potential plastic region as detailed in 2.6.5.5.

2.6.5.5 *Likely maximum material strengths*

A2 | Overstrength actions in potential plastic regions shall be determined assuming the appropriate cross section of the member and material overstrengths as set out below:

(a) The stress resisted by reinforcement shall be taken as $\phi_{o,fy}$ times design yield strength of reinforcement, f_y. For reinforcement which complies with AS/NZS 4671, $\phi_{o,fy}$ shall be taken as 1.25 for Grade 300 and 1.35 for Grade 500;

(b) The stress in flexural reinforcement given in (a) above shall be used unless other values supplied by the manufacturer can be shown to be appropriate after peer reviewed special studies;

(c) The compression strength of concrete shall be taken as $\left[f'_c + 15\right]$ MPa.

2.6.5.6 *Ends of columns*

For potential inelastic regions in a column, which can form against a base slab or other members that effectively confine the compression region, the overstrength bending moment, M_o^* shall be calculated
A2 | taking into account axial compression load as given in Equation 2–10. In no case shall the overstrength moment be taken as less than the value defined in 2.6.5.5.

A2 |
$$M_o^* = \left(\phi_{o,fy} + 2\left(\frac{N_o^*}{f'_c A_g} - 0.1 \right) \right)^2 M_n \quad \text{.....(Eq. 2–10)}$$

where

$\phi_{o,fy}$ = 1.25 for Grade 300 reinforcement

= 1.35 for Grade 500 reinforcement

N_o^* is the axial load that acts concurrently with M_o^*.

2.6.5.7 *Capacity design for regions outside potential plastic regions*

Where the design strengths for regions outside the potential plastic regions are determined on the basis of actions, which can be transmitted to them through potential plastic regions, a strength reduction factor that is equal to or less than 1.0 shall be used. In assessing these design actions allowance shall be made for:

(a) The most adverse combination of overstrength actions in the potential plastic regions which may be transmitted into the member for the action being considered;

(b) Gravity loads which may act on the member;

(c) The change in dynamic behaviour of the structure changing the distribution of moments and shear forces (see Appendix D for dynamic magnification factors or defined distributions of actions);

(d) An analysis which includes vertical seismic actions shall be made for sensitive horizontal members such as cantilever beams or slabs containing pretensioned members. A2

2.6.5.8 *Concurrency and capacity design*

In capacity design the effects of seismic actions occurring simultaneously along two axes at right angles shall be considered in the detailing of members, which are part of two-way horizontal force-resisting systems.

Columns and walls, including their joints and foundations, which are part of a two-way horizontal force-resisting system, with structural elements aligned along two axes, shall be detailed to sustain the concurrent actions as defined in (a) and (b) below:

(a) Overstrength bending moments and shears, amplified by dynamic magnification for one axis together with the overstrength actions from the other axis (ω = 1.0) with both possible combinations being considered for the two axes;

(b) Critical axial force found assuming concurrent yielding of all beams framing into the column, modified where appropriate, as defined in Appendix D, to allow for the limited number of plastic hinges which develop simultaneously on different levels of a multi-storey structure.

2.6.5.9 *Transfer diaphragms*

Floor and roof systems in buildings shall be designed to act as horizontal structural elements, where required, to transfer seismic forces to frames or structural walls in accordance with Section 13.

2.6.6 *Additional requirements for nominally ductile structures*

2.6.6.1 *Limitations for nominally ductile structures*

Nominally ductile structures shall be proportioned to ensure that when they are subjected to the seismic load combinations specified in AS/NZS 1170:Part 0 for the ultimate limit state or other referenced loading standard, the following conditions are satisfied.

(a) When the structural system is such that under seismic actions larger than anticipated, mechanisms could only develop in the same form as those permitted by 2.6.7 for ductile structures, or those of limited ductility, the selected structure is exempt from the additional seismic requirements of all sections of this Standard.

(b) When a mechanism could develop in a form which is not permitted for a ductile or limited ductile structures, the relevant mechanism or mechanisms shall be identified. Potential plastic hinge regions shall be identified, and detailed for ductile or limited ductile plastic regions such that the material strain limits given in 2.6.1.3 are not exceeded, in accordance with the additional seismic design requirements of this Standard.

2.6.7 *Additional requirements for ductile frames and limited ductile moment resisting frames*

2.6.7.1 *Ductile and limited ductile moment resisting frames*

The requirements of 2.6.7 shall be satisfied for frames forming part of the primary lateral load resisting system. Frames that are secondary structural elements shall satisfy 2.6.10.

2.6.7.2 *Acceptable column sidesway mechanisms*

Column sidesway mechanisms may be used as a design solution for:

(a) The top storey of any moment resisting frame;

(b) Frames not exceeding two storeys where the columns are detailed using the ductile provisions of 10.4;

(c) Bridge piers.

In all other cases the sidesway mechanism shall be based on the beam sway mode.

2.6.7.3 *Beam design for column sidesway structures*

Where the requirements of 2.6.7.2 are satisfied, and capacity design procedures mean that yielding of the beams is unlikely, the beams shall be detailed as specified in 9.3.

2.6.7.4 *Alternative design methods for columns in multi-storey frames*

Columns in ductile or limited ductile moment resisting frames shall be designed to have a high level of protection against the formation of a non-ductile failure mechanism in a major earthquake. Method A or Method B, as detailed in Appendix D shall be used to determine the critical design actions to achieve this objective.

2.6.7.5 *Design actions in columns*

When determining the design actions in columns:

(a) The axial load at critical sections shall be determined from the self weight of columns and attachments to the columns, gravity load shear forces and shear forces induced in the beams due to overstrength moments acting in the plastic hinge regions. In assessing the critical axial load level at a section, the axial load induced by all the beams framing into the column above the section being considered shall be included (see Appendix D).

(b) The nominal flexural strength of the column shall be equal to or greater than that required to sustain overstrength moments that act on the column from all beams intersecting the column amplified by appropriate dynamic magnification factors. Where a column acts in two moment resisting frames it shall be designed to sustain the moments applied simultaneously by the beams in the frames, amplified as required in 2.6.5.8.

(c) The columns shall be designed to sustain the critical shear forces transmitted to the columns by all the beams framing into the column above the section being considered (see appendix D).

A2 (d) For columns in two-way frames; design moments at primary plastic hinges, and immediately below the uppermost level in the frame, shall be determined from the critical ultimate limit state load combination, including seismic load combinations where 100 % of the seismic load on one axis is applied simultaneously, with 30 % of the seismic load on a second axis perpendicular to the first axis.

2.6.8 ***Ductile walls and dual structures***

2.6.8.1 *Inelastic deformation of structural walls*

All structural walls, which are designed to provide lateral force resistance, shall be designed to dissipate energy by flexural yielding at the ultimate limit state.

2.6.8.2 *Shear strength of structural walls*

In providing the shear strength of a structural wall in the ultimate limit state, allowance shall be made in the shear force envelope for flexural overstrength and dynamic effects.

2.6.8.3 *Coupled walls*

When two or more walls are interconnected by substantial ductile beams, part of the seismic energy to be dissipated in the ultimate limit state shall be assigned to the coupling system. Capacity design procedures shall be used to ensure that the ductility of the coupling system can be maintained at its overstrength value.

2.6.8.4 *Ductile dual structures*

Where a combination of different lateral force-resisting structural elements is used in a structure, rational analysis shall be employed, taking into account the relative stiffness and location of elements, to allocate the seismic resistance to each element. Where diaphragms are required to transfer seismic forces between elements the design shall allow for the actions associated with overstrength of the elements.

2.6.9 *Structures incorporating mechanical energy dissipating devices*

The design of structures incorporating flexible mountings and mechanical energy dissipating devices is acceptable provided that the following criteria are satisfied at the ultimate limit state:

(a) The performance of the devices used is substantiated by tests;

(b) Proper studies are made towards the selection of suitable design earthquakes for the structure;

(c) The degree of protection against yielding of the structural members is at least as great as that implied in this Standard relating to the conventional seismic design approach without energy dissipating devices;

(d) The structure is detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

2.6.10 *Secondary structural elements*

2.6.10.1 *Definitions*

Secondary structural elements are those which at the ultimate limit state do not form part of the primary seismic action resisting system, but which are subjected to actions due to accelerations transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 are those which are subjected to inertia forces but which, by virtue of their detailed separations, are not subjected to forces induced by the deformation of the supporting primary elements or secondary elements of Group 2;

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia forces, as for Group 1, and to forces induced by the deformation of the primary elements.

2.6.10.2 *Group 1 secondary elements*

Group 1 elements shall be detailed for separation to accommodate deformations where the ultimate limit state lateral deflections of the primary seismic force-resisting system, calculated as specified in AS/NZS 1170 and NZS 1170.5, or other referenced loading standard are reached. Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, and, where appropriate, allow for deformation due to other loading conditions such as gravity loading. For elements of Group 1:

(a) The inertia force, F_{ph}, used in the design shall be that specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard;

(b) Detailing shall be such as to allow ductile behaviour if necessary and in accordance with the assumptions made in the analysis. Fixings for non-structural elements shall be designed and detailed in accordance with 17.6.

2.6.10.3 *Group 2 secondary elements*

Group 2 elements shall be detailed to allow ductile behaviour if necessary where the ultimate limit state lateral deflections of the primary seismic force-resisting system, calculated as specified in AS/NZS 1170 and NZS 1170.5 or other referenced loading standard, are reached for elements of Group 2:

(a) Additional seismic requirements of this Standard need not be satisfied when the design forces are derived from the imposed ultimate limit state lateral deflection, and analysis indicates that the element does not sustain inelastic deformation;

(b) Additional seismic requirements of this Standard shall be met when inelastic behaviour is assumed to occur at levels of deformation below the ultimate limit state lateral deflection;

(c) The inertia force, F_{ph}, shall be that specified by AS/NZS 1170 and NZS 1170.5;

(d) Forces induced by the deformation of the primary elements shall be those arising from the calculated ultimate limit state lateral deflection having due regard to the pattern and likely simultaneity of deformations;

(e) Analysis shall be by any rational method in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-quarter of the above calculated ultimate limit state lateral deflection of the primary elements, as specified in AS/NZS 1170 and NZS 1170.5.

A2 (f) Where inelastic deformation is required, potential plastic regions shall be identified and detailed to meet the requirements of this Standard. Plastic regions may be located in columns.

3 DESIGN FOR DURABILITY

3.1 Notation

3.1.1 *Symbols*

f'_c specified compressive strength of concrete, MPa

3.1.2 *Abbreviations*

FA Fly ash – AS 3582:Part 1 Supplementary cementitious materials for use with Portland and blended cement
GB General purpose blended cement – NZS 3122
GP General purpose Portland cement – NZS 3122
HE High early strength cement – NZS 3122
MS Amorphous silica – AS/NZS 3582:Part 3
SCM Supplementary cementitious material
GBS Ground granulated iron blast-furnace slag – AS 3582 Part 2

3.2 Scope

3.2.1 *Concrete*

The provisions of this section shall apply to the detailing and specifying for durability of plain, reinforced and prestressed concrete members with f'_c ranging from 20 MPa to 100 MPa and a design life of 50 years, or for a limited range of conditions, 100 years. A2

3.2.2 *Cementitious binders*

Durability design to this Standard shall be based on the use of concrete made with GP, GB or HE cement complying with NZS 3122 with or without supplementary cementitious materials complying with AS 3582.

3.2.3 *Design considerations*

Durability shall be allowed for in design by determining the exposure classification in accordance with 3.4 and, for that exposure classification, complying with the appropriate requirements for:

(a) Concrete quality and curing, in accordance with 3.5 to 3.12;
(b) Cover in accordance with 3.11 or 3.12;
(c) Chemical content restrictions, in accordance with 3.14;
(d) Alkali silica reaction precautions in accordance with 3.15;
(e) Protection of fixings to 3.13.

3.2.4 *Design for particular environmental conditions*

In addition to the requirements specified in 3.2.3:

(a) Members subject to aggressive soil and ground water shall satisfy the requirements of 3.5;
(b) Members subject to abrasion shall satisfy the requirements of 3.9;
(c) Members subject to cycles of freezing and thawing shall satisfy the requirements of 3.10.

3.3 Design life

3.3.1 *Specified intended life*

The provisions of this section shall apply to the detailing and specifying for durability of reinforced and prestressed concrete structures and members with a specified intended life of 50 or 100 years. Compliance with this section will ensure that the structure is sufficiently durable to satisfy the requirements of the NZ Building Code throughout the life of the structure, with only normal maintenance and without requiring reconstruction or major renovation. The 50 years corresponds to the minimum structural performance life of a member to comply with that code.

3.4 Exposure classification

3.4.1 *General*

A2 Where concrete will be in wet or saline conditions, aggressive soil or groundwater, or in contact with harmful industrial materials or processes, appropriate measures shall be highlighted in the drawings and specifications to ensure the durable integrity of the structure.

3.4.2 *Environmental exposure classification*

3.4.2.1 *Exposure classification categories*

The exposure classification for a surface of a steel reinforced or prestressed member shall be determined from Table 3.1. Except for categories 4(b) and 5, this table need not apply to unreinforced members, members with non-metallic reinforcement, or steel fibre concrete provided that such concrete does not contain metals that rely on the concrete for protection against environmental degradation.

Table 3.1 – Exposure classifications

Surface and exposure environment	Exposure classification
1 Surfaces of members in contact with the ground:	
(a) Protected by a damp proof membrane	A1
(b) In non-aggressive soils	A2
2 Surfaces of members in interior environments:	
(a) Fully enclosed within a building except for a brief period of weather exposure during construction (1)	A1
(b) In buildings or parts thereof where the members may be subject to repeated wetting and drying (1)	B1
3 Surfaces of members in above-ground exterior environments in areas that are:	
(a) Inland (2)	A2
(b) Coastal perimeter (2)	B1
(c) Coastal frontage (see 3.4.2.4)	B2
4 Surfaces of members in water: (3)	
(a) (i) In fresh (not soft) water contact	B1
(ii) In fresh (not soft) water pressure	B2
(iii) In fresh (not soft) water running	B2
(b) (i) In fresh (soft) water contact	B2
(ii) In fresh (soft) water pressure	U
(iii) In fresh (soft) water running	U
(c) In sea water:	
(i) Permanently submerged	B2
(ii) Tidal/splash/spray (see 3.4.2.5)	C
5 Surfaces of members exposed to chemical attack (see 3.4.3) in:	
(a) Slightly aggressive chemical environment	XA 1
(b) Moderately aggressive chemical environment	XA 2
(c) Highly aggressive chemical environment	XA 3
6 Surfaces of members in other environments: Any exposure environment not otherwise described in items 1–5 above.	U

NOTE –

(1) Where concrete is used in industrial applications, consideration shall be given to the effects of any manufacturing process on the concrete which may require a reclassification to exposure classification U. (See 3.8)

(2) The boundary between the different exterior environments is dependent on many factors which include distance from sea, prevailing wind and its intensity.

(3) Water analysis is required to establish the characteristics of water softness.

3.4.2.2 *Mixed exposures*

For determining concrete quality requirements in accordance with 3.5 to 3.12, as appropriate, the exposure classification for the member shall be taken as the most severe exposure of any of its surfaces.

3.4.2.3 *Individual surfaces*

For determining cover requirements for corrosion protection in accordance with 3.11, the exposure classification shall be taken as the classification for the surface from which the cover is measured.

3.4.2.4 *Coastal frontage zone extent*

The extent of the coastal frontage zone shall be determined by reference to Table 3.2(b). General wind directions are indicated in Figure 3.1 (a) to (f). As an alternative solution, a site-specific evaluation can be made. The extent will depend on winds, wave action and topography. More specific wind frequency data can be obtained from the National Institute of Water and Atmospheric Research Ltd. | A2

3.4.2.5 *Tidal/splash/spray zone*

The extent of the C tidal/splash/spray zone is given in Table 3.2(b). As an alternative a site - specific evaluation of spray drift can be made taking into account wind strength, wave action and local topography. Structures over the sea of body of saline water where breaking waves occur shall be classification C. | A2

The boundary of the C zone and the B2 zone in the vertical direction shall be taken as mean low water level at depth and shall be determined from prevailing wind and sea conditions for height above sea level.

Table 3.2(a) – Prevailing or common winds | A2

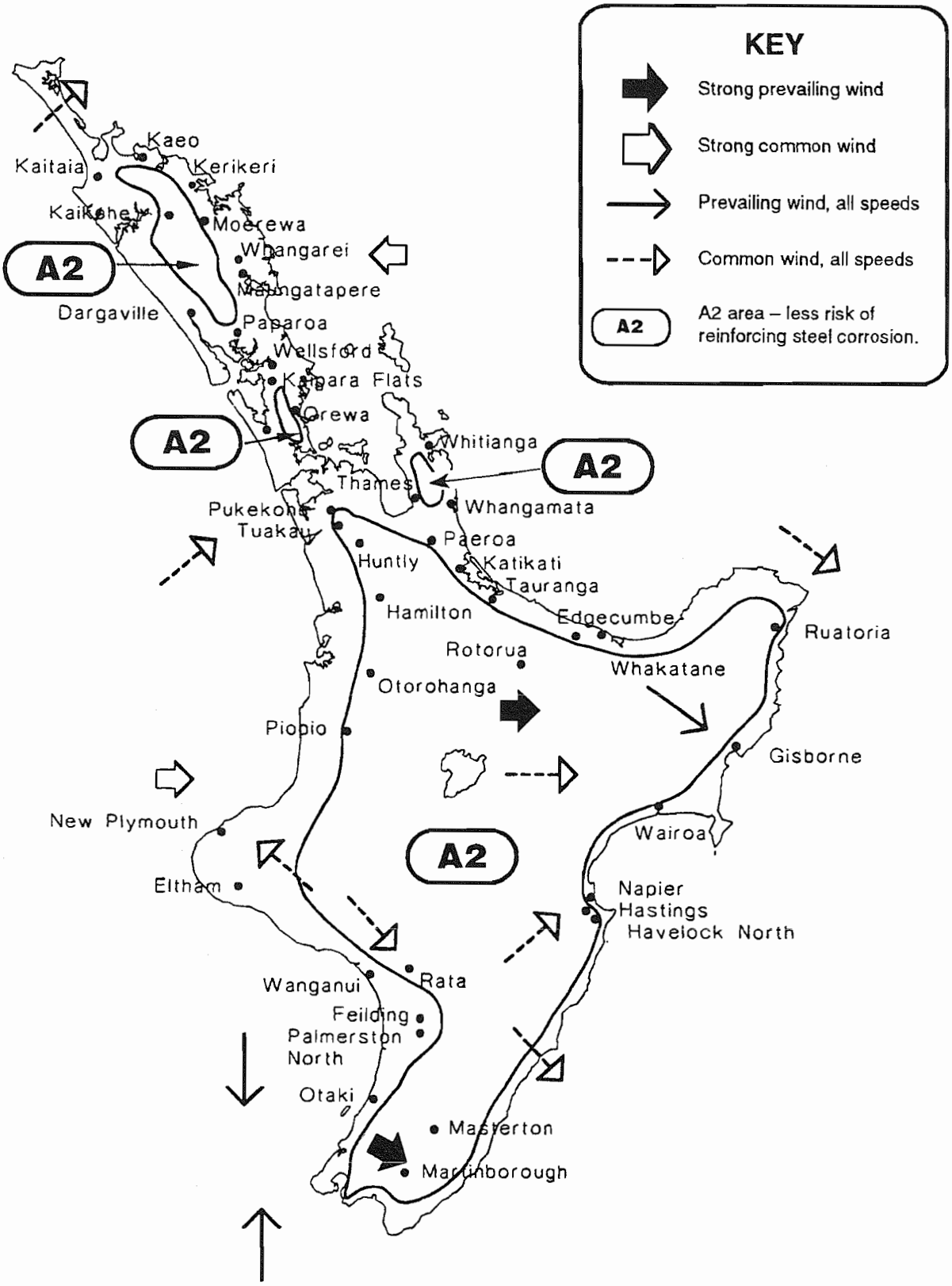
Locality	Direction prevailing or common wind comes from
Auckland	Southwest
Wellington	North, South or Northwest
Christchurch, Dunedin	Northwest, Northeast or Southwest
Other localities	Refer to Figure 3.1 (a) and (b)

Table 3.2(b) – Definition of B2 and C zones

Direction from coast to site exposed to common or prevailing wind	B2 (Coastal frontage)	C (Tidal / Splash / Spray)
Downwind	Between 30 m and 500 m inland of the high tide mark	Offshore and up to 30 m inland of the high tide mark
Directions other than downwind	From the high tide mark to 100 m inland	Offshore and up to the high tide mark

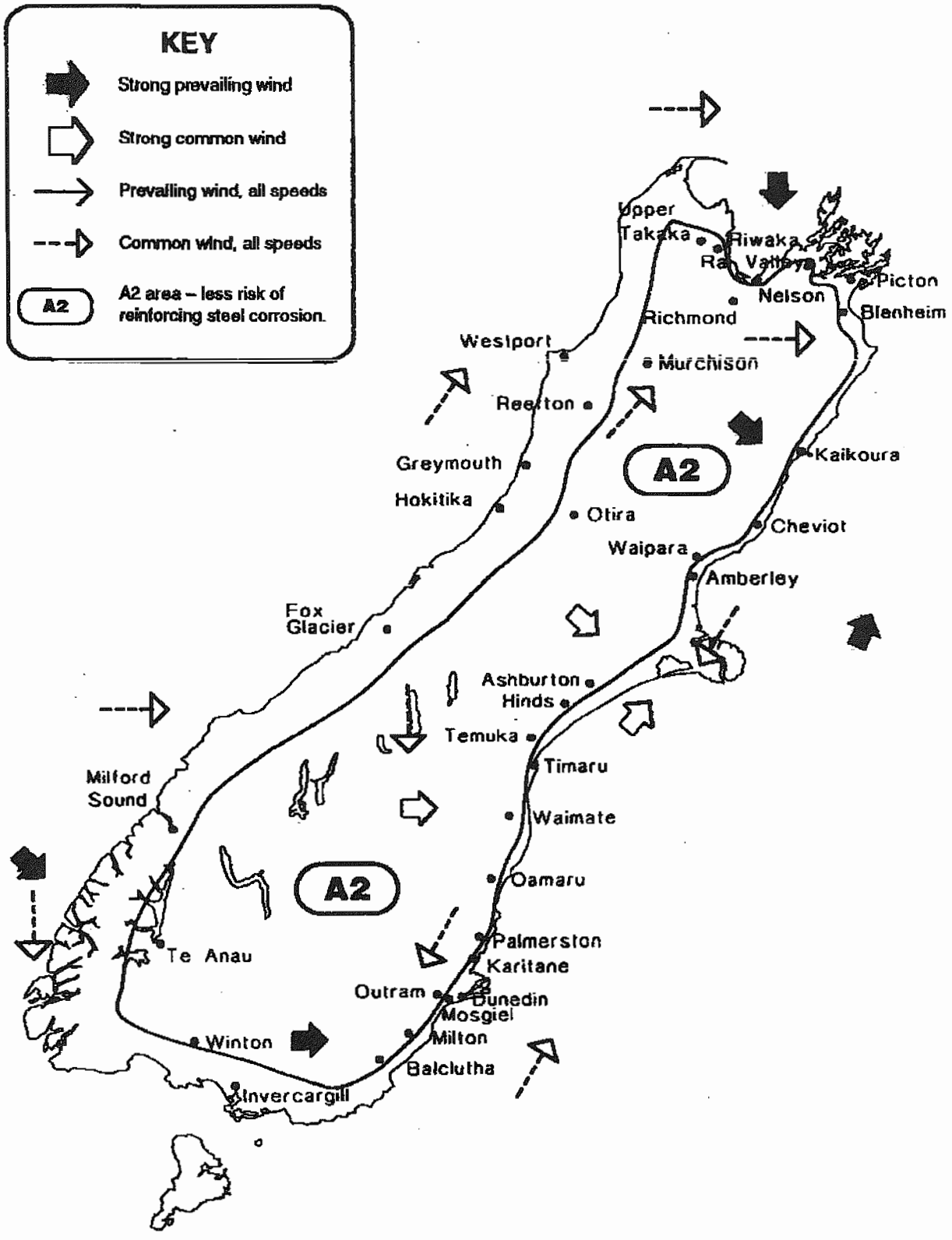
3.4.2.6 *Boundary between coastal perimeter and inland zones*

Figure 3.1(a) to (f) indicate the boundary between the inland (A2) and the coastal perimeter (B1) exposure classifications.



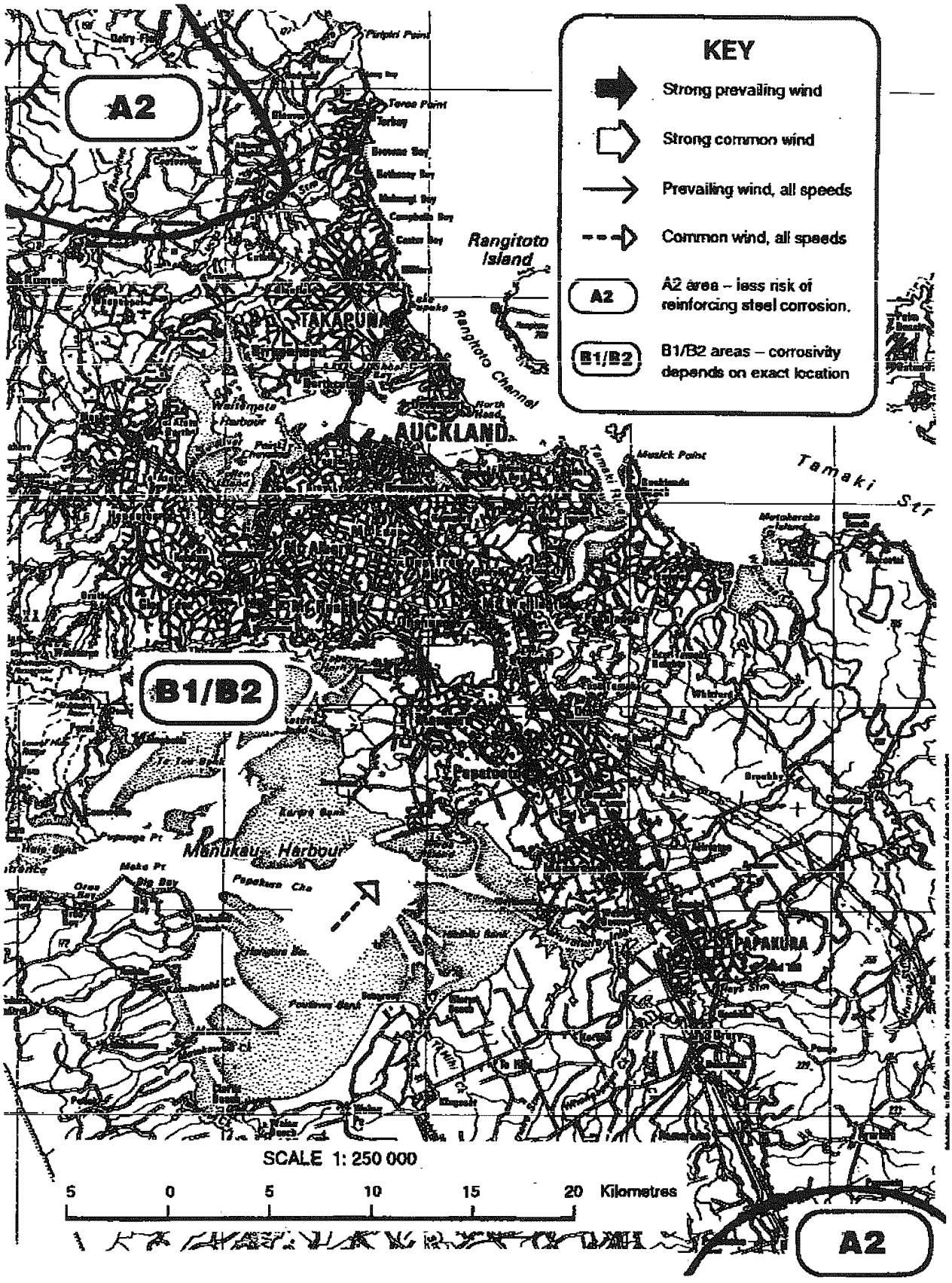
(a) North Island

Figure 3.1 – Exposure classification maps



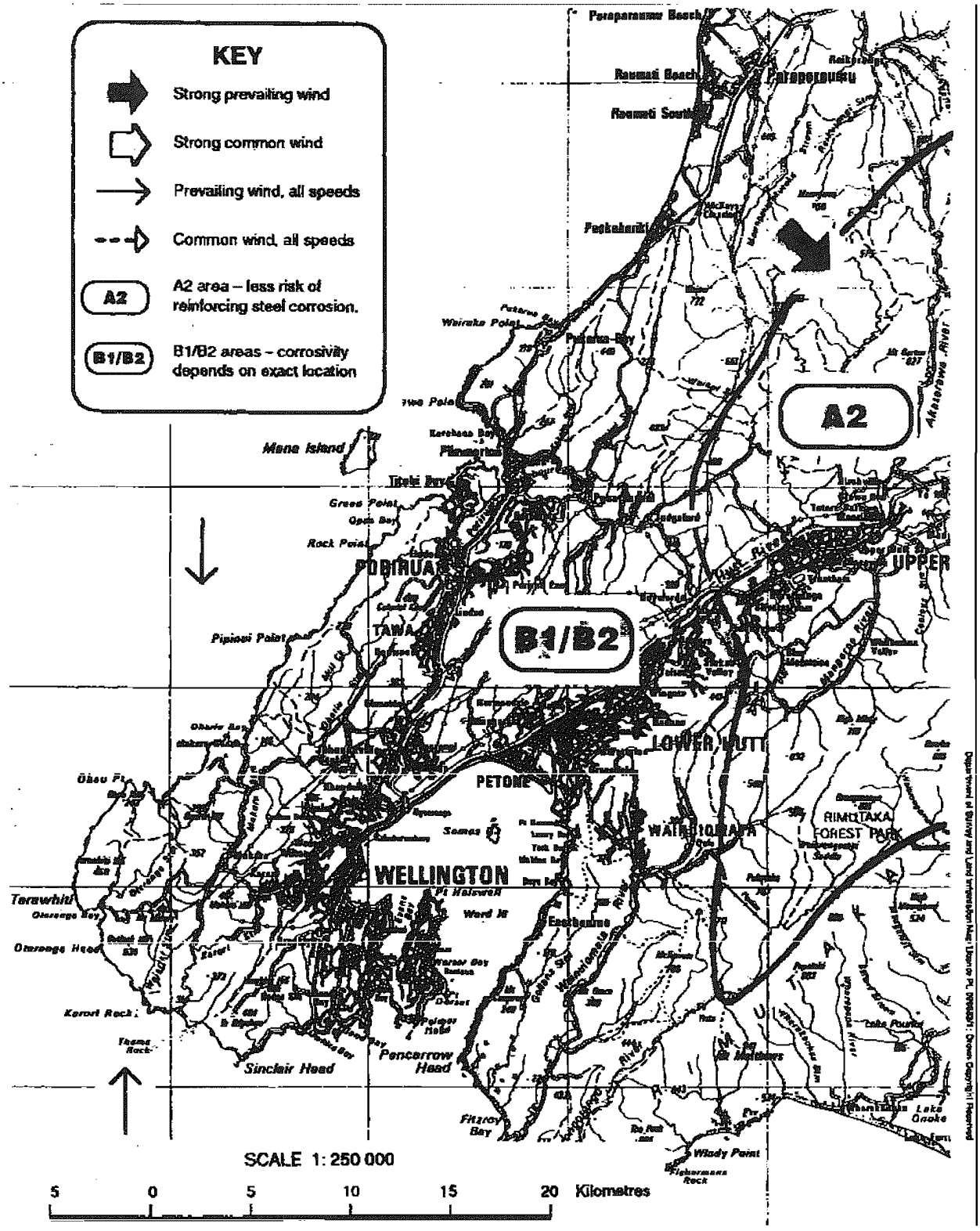
(b) South Island

Figure 3.1 – Exposure classification maps (continued)



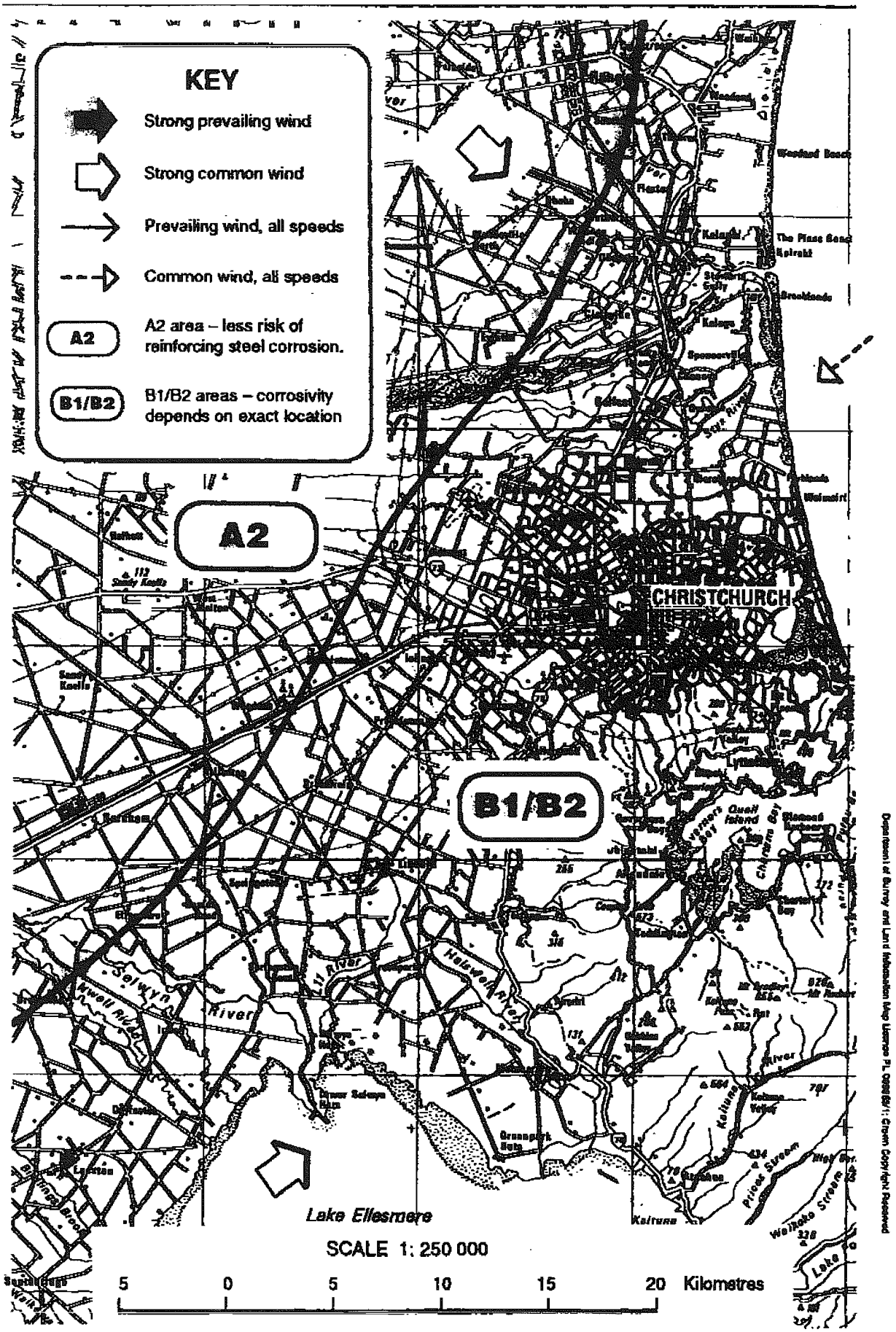
(c) Auckland

Figure 3.1 – Exposure classification maps (continued)



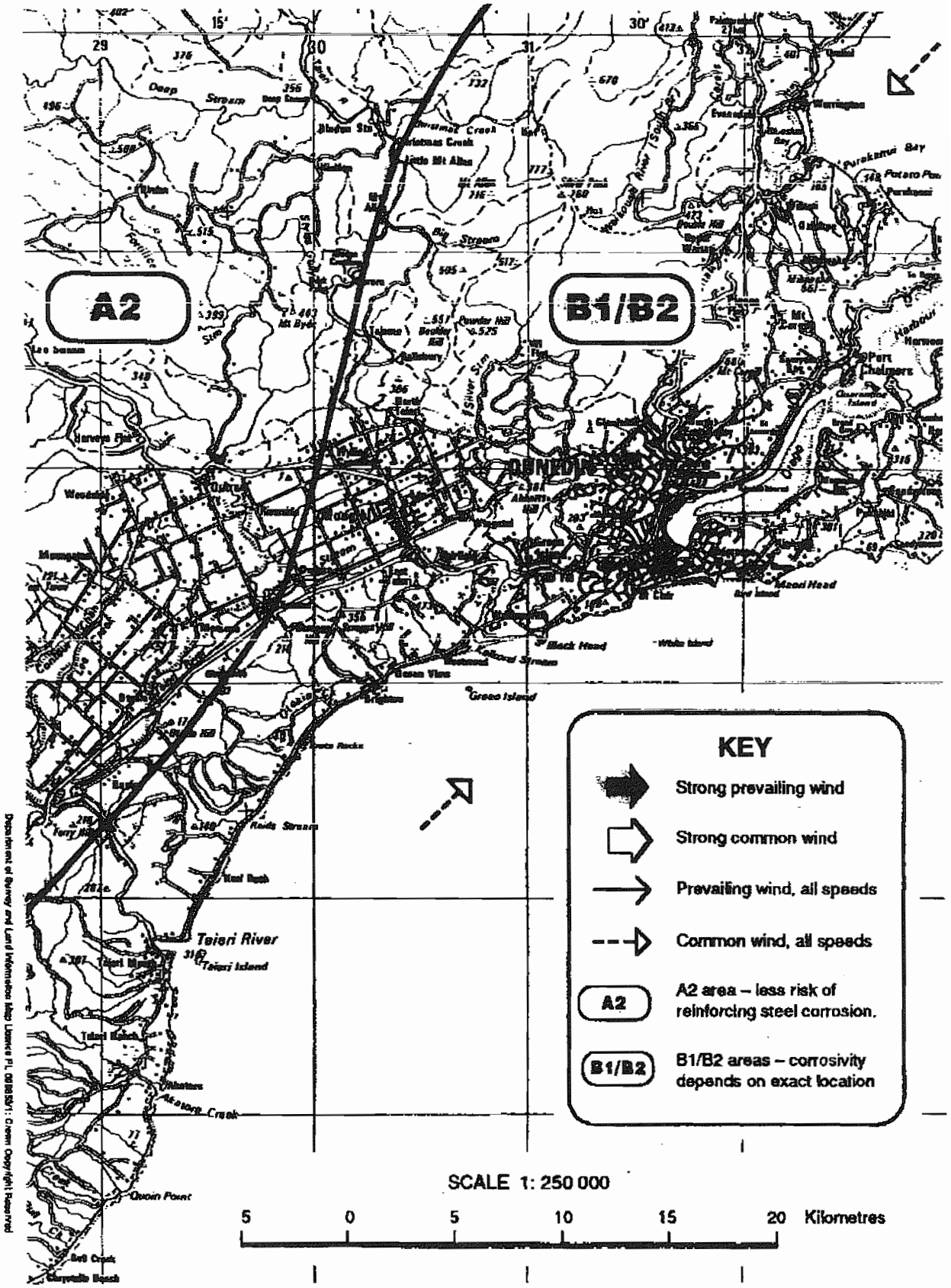
(d) Wellington

Figure 3.1 – Exposure classification maps (continued)



(e) Christchurch

Figure 3.1 – Exposure classification maps (continued)



(f) Dunedin

Figure 3.1 – Exposure classification maps (continued)

3.4.3 *Chemical exposure classification*

3.4.3.1 *Chemical attack from natural soil and groundwater*

The chemical exposure classification for a surface of a concrete member exposed to chemical attack from natural soil and ground water shall be determined from Table 3.3. The degree of aggressivity may be underestimated in cases where combined attacks plus high temperature and high relative humidity occur (e.g. geothermal areas). In this case the next higher degree of aggressivity determines the chemical exposure classification, unless a special study proves otherwise.

A2

Table 3.3 – Guide for exposure classification for chemical attack of concrete from natural soil and groundwater

Chemical exposure classification	Chemical constituents[1]				
	Ground water [2][3]		Soil [3][4]		
	pH	Sulphate SO_4^{2-} (mg/ℓ)	Acidity[5] (ml/kg of air dry soil)	Acid soluble sulphate SO_4^{2-} [6] (% of air dry soil passing a 2 mm test sieve)	Water soluble sulphate SO_4^{2-}[6] (g/ℓ in 2:1 water soil extract)
XA1	6.5 – 5.5	200 – 600	>200 (Baumann-Gully)	0.2 – 0.3	0.6 – 1.8
XA2	5.5 – 4.5	600 – 3000	–	0.3 – 1.2	1.8 – 3.7
XA3	4.5 – 4.0	3000 – 6000	–	1.2 – 2.4	3.7 – 6.7

NOTE –

(1) Magnesium content is considered to be less than 1000 mg/ℓ.
(2) Mobility of water is considered to be in an approximately static condition.
(3) Soil and groundwater temperature 5° C to 25° C.
(4) Nominally dry sites or soils with permeability less than 10^{-5} m/s (e.g. unfissured clay) may be moved into a lower class.
(5) The Baumann-Gully acidity is expressed as volume of 0.1 mol/litre sodium hydroxide required to neutralise acetic acid, in ml/kg of air dried soil (DIN 4030-2).
(6) Measure either acid soluble or water soluble sulphate in accordance with BS 1377-3, depending on which is more appropriate for the site being assessed. Sulphate results expressed in terms of SO_3 shall be multiplied by 1.2 to convert them to SO_4^{2-} values.
(7) A special study under exposure classification U is required where there is:
  (a) Limits outside of Table 3.3 or Table 3.3 Notes;
  (b) Direct contact with chemically aggressive environments; or
  (c) High water velocities and/or water under pressure in combination with the aggressive agents stated in Table 3.3. In these circumstances consult BS 8500-1 for guidance.

3.4.3.2 *Other chemical attack*

An acidity represented by a pH of 5.0 to 5.5 may be considered as a practical limit of tolerance of high quality concrete in contact with any acids. For pH lower than 5.0, the environment shall be assessed as exposure classification U.

3.5 Requirements for aggressive soil and groundwater exposure classification XA

Concrete in members subject to chemical attack shall be specified in accordance with Table 3.4. Such concrete shall be specified as 'Special Concrete' under NZS 3109 Clause 6.3.

Table 3.4 – Requirements for concrete subjected to natural aggressive soil and groundwater attack for a specified intended life of 50 years

Chemical exposure classification	Max. water cementitious ratio	Min. cover (mm)	Min. binder content (kg)	Additional requirement
XA1	0.50	50	340	—
XA2	0.45	50	370	SCM
XA3	0.40	55	400	SCM

NOTE –

(1) Binders containing combinations of cement and supplementary cementitious materials (SCM) (30 % fly ash, 65 % slag or 8 % amorphous silica) provide significantly increased resistance to chemical attack mechanisms.

(2) Where low pH and high exchangeable soil acid conditions prevail, an additional protection (e.g. protective coating, or other form of physical protection) may be required. This may allow for reduction of originally specified concrete parameters.

3.6 Minimum concrete curing requirements

Minimum concrete curing requirements for the exposure classifications given in Table 3.1 are given in Table 3.5:

Table 3.5 – Minimum concrete curing requirements

Exposure classification	Curing period[(1) (3)] (under ambient conditions)
A1, A2, B1	3 days
B2	7 days
C	7 days[(2)]
XA1	3 days
XA2	7 days[(2)]
XA3	7 days[(2)]

NOTE –

(1) Curing shall comply with Clause 7.8 of NZS 3109.

(2) Concrete in C, XA2, and XA3 zones shall be cured continuously by direct water application such as ponding or continuous sprinkling, or by continuous application of a mist spray.

(3) Alternative curing methods may be used, provided a special study proves that the alternative method provides concrete durability performance equivalent to that provided by direct water application.

A2

3.7 Additional requirements for concrete exposure classification C

3.7.1 *Supplementary cementitious materials*

The concrete shall contain a supplementary cementitious material. Table 3.6 and Table 3.7 give three compliant options using three different supplementary cementitious materials.

3.7.2 *Water/binder ratio and binder content*

Binder combination, minimum binder material content and maximum water/binder ratio shall be specified in addition to compressive strength to comply with Table 3.6 and Table 3.7.

3.7.3 *Special concrete*

Concrete to be used in exposure classification C shall be specified as 'Special Concrete' under NZS 3109 Amendment No.1, August 2003 Clause 6.3. The quality control requirements for the concrete supply, and any special durability related testing shall be ascertained between the specifier and the concrete producer.

A2

Table 3.6 – Minimum required cover for a specified intended life of 50 years

Exposure classification	Cement binder type	Specified compressive strength f'_c (MPa)							
		20	25	30	35	40	45	50	60 - 100
		Minimum required cover (mm)							
A1	GP, GB or HE	25	25	20	20	20	20	20	20
A2	GP, GB or HE	40	35	30	30	25	25	25	20
B1	GP, GB or HE	50	40	35	35	30	30	30	25
B2	GP, GB or HE	-	-	45	40	35	30	30	25
C (1)	30 % FA	-	-	-	-	60	60	60	55
C (1)	65 % GBS	-	-	-	-	-	50	50	50
C (1)	8 % MS	-	-	-	-	-	60	50	50

NOTE –
(1) For zone C the total binder content shall be equal to or greater than 350 kg/m^3, and water to binder ratio shall not exceed 0.45.
(2) The minimum cover for the C zone shall be 50 mm.

Table 3.7 – Minimum required cover for a specified intended life of 100 years

Exposure classification	Cement binder type	Specified compressive strength f'_c (MPa)						
		25	30	35	40	45	50	60 – 100
		Minimum required cover (mm)						
A1	GP, GB or HE	35	30	30	30	30	30	25
A2	GP, GB or HE	50	40	40	35	35	35	30
B1	GP, GB or HE	55	50	45	40	40	35	30
B2	GP, GB or HE	-	65	55	50	45	40	35
C (1)	30 % FA	-	-	-	-	70	60	60
C (1)	65 % GBS	-	-	-	-	60	50	50
C (1)	8 % MS	-	-	-	-	-	50	50

NOTE –
(1) For zone C the total binder content shall be equal to or greater than 350 kg/m^3 and water to binder ratio shall not exceed 0.45.
(2) The minimum cover for the C zone shall be 50 mm.

3.8 Requirements for concrete for exposure classification U

Exposure Classification U represents an exposure environment not specified in Table 3.1 for which the degree of severity of exposure should be assessed by the designer. Concrete in members subject to exposure classification U shall be specified to ensure durability under the particular exposure environment and for the chosen design life. Protective coatings may be taken into account in the assessment of concrete requirements.

3.9 Finishing, strength and curing requirements for abrasion

3.9.1 *Abrasion from traffic*

Concrete for members subject to abrasion from traffic shall comply with the specified compressive strength and construction requirements given in Table 3.8.

Table 3.8 – Requirements for abrasion resistance for a specified intended life of 50 years

<table>
<tr><th>Class</th><th>Service conditions</th><th>Application</th><th>Finishing process</th><th>Curing</th><th>Minimum specified compressive strength f'_c (MPa)</th></tr>
<tr><td>Special</td><td>Severe abrasion and impact from steel or hard plastics wheeled traffic or scoring by dragged metal objects</td><td>Very heavy duty engineering workshops and very intensively used warehouses</td><td colspan="3" rowspan="2">Special flooring techniques may be used.
The suitability of concrete flooring for this class should be established with the manufacturer or flooring contractor</td></tr>
<tr><td>AR1</td><td>Very high abrasion: steel or hard plastics wheeled traffic and impact</td><td>Heavy duty industrial workshops and intensively used warehouses</td></tr>
<tr><td>AR2</td><td>High abrasion: steel or hard plastics wheeled traffic</td><td>Medium duty industrial and commercial</td><td rowspan="2">Power floating and at least two passes with a power trowel</td><td rowspan="2">7 days water curing using ponding or covering; or the use of a curing membrane that meets NZS 3109</td><td>40 MPa</td></tr>
<tr><td>AR3</td><td>Moderate abrasion: Rubber tyred traffic</td><td>Light duty industrial and commercial</td><td>30 MPa</td></tr>
<tr><td colspan="3">Commercial and industrial floors not subject to vehicular traffic</td><td>As nominated by the designer</td><td>3 days minimum</td><td>25 MPa</td></tr>
</table>

3.9.2 *Abrasion by waterborne material*

Abrasion erosion damage caused by the abrasive effects of waterborne sediment (i.e. silt, sand, gravel, rock) and other debris impinging on a concrete surface may affect structures such as spillways, culverts and bridge piers. For guidance on materials and techniques to control abrasion erosion refer to erosion of Concrete in Hydraulic Structures", Report by ACI Committee 210, Report No. ACI 210R-93, American Concrete Institute, Michigan, USA, 1993..

3.10 Requirements for freezing and thawing

In addition to the other durability requirements of this section, where a surface may be exposed to cycles of freezing and thawing, concrete in the member shall:

(a) Contain a percentage of entrained air within the following ranges for:
   (i) 10 mm to 20 mm nominal size aggregate.....4 % to 8 % ;
   (ii) Greater than 20 mm nominal size aggregate.....3 % to 6 %;
   where the percentage of entrained air is determined in accordance with NZS 3112:Part 1 and

(b) Have a specified compressive strength, f'_c, equal to or greater than:
   (i) 30 MPa for frequent exposure (≥ 50 cycles per year);
   (ii) 25 MPa for occasional exposure (25 – 49 cycles per year).

3.11 Requirements for concrete cover to reinforcing steel and tendons

3.11.1 *General*

3.11.1.1 *Cover*

The cover to reinforcing steel and tendons shall be the greater of the values determined from 3.11.2 and 3.11.3, as appropriate, unless greater covers are required by Section 4 for fire resistance. Cover shall be measured to the stirrups or reinforcement which is closest to the surface of the member.

3.11.1.2 *Effect of crack width control on cover*

Crack width control in accordance with 2.4.4 may limit maximum covers allowable for durability purposes.

3.11.2 *Cover of reinforcement for concrete placement*

3.11.2.1 *Reinforcing steel configuration*

The cover and arrangement of the steel shall be such that concrete can be properly placed and compacted in accordance with NZS 3109. The spacing requirements are given in Clause 8.3.

3.11.2.2 *Minimum cover*

The cover shall be equal to or greater than either:

(a) The maximum nominal aggregate size for Exposure Classifications A1 and A2 and 1.25 times the maximum nominal aggregates size for other exposure zones; or

(b) The nominal size of bar or tendon to which the cover is measured;

whichever is the greater.

3.11.3 *Cover for corrosion protection*

3.11.3.1 *General*

For corrosion protection, the cover shall be equal to or greater than the appropriate value given in 3.11.3.2 and 3.11.3.3.

3.11.3.2 *Formed or free surfaces*

Where concrete is compacted in accordance with NZS 3109; cover to formwork complying with NZS 3109, or to free surfaces shall be equal to or greater than the value given in Table 3.6 or Table 3.7 appropriate to the design life, the exposure classification and specified concrete strength. Table 3.6 and Table 3.7 give cover requirements for a specified intended life of 50 years and 100 years respectively.

3.11.3.3 *Casting against ground*

Where concrete is cast on or against ground and compacted in accordance with NZS 3109, the minimum cover for a surface in contact with the ground shall be 75 mm, or 50 mm if using a damp-proof membrane between the ground and the concrete to be cast.

3.12 Chloride based life prediction models and durability enhancement measures

3.12.1 *The use of life prediction models*

Life prediction models can be used as an alternative to Table 3.6 and Table 3.7 for the C zone and B2 zone, however they are outside the scope of this Standard. The tables will generally provide solutions which are more conservative than those derived from the use of a model. Guidance on the use of life prediction models to determine cover as an alternative to Table 3.6 and Table 3.7, is given in the commentary.

3.12.2 *Other durability enhancing measures*

There are a number of durability enhancing measures which can be taken to extend the life of concrete structures beyond those determined in accordance with 3.11.3.2. These include concrete coatings, corrosion inhibiting admixtures, galvanised or stainless steel reinforcement, controlled permeability formwork and GRC permanent formwork.

3.13 Protection of cast-in fixings and fastenings

3.13.1 *Fixing and fastening protection*

Where metallic fixings or fastenings are exposed in the finished structure, or where the cover to any part of the fixing is less than that required by 3.11 for the particular exposure classification, additional protection shall be provided in accordance with Table 3.9.

Table 3.9 – Protection required for steel fixings and fastenings for a specified intended life of 50 years

A2

Exposure classification	Material/protection
Dry, internal location, not subject to airborne salts or rain wetting	
A1	Mild steel (uncoated non-galvanised) Electroplated zinc anchors (1)
Open to airborne salts, but not rain washed	
A2, B1	Hot-dip galvanised steel (2) Hot-dip galvanised anchors(2)
B2(3), C	Stainless steel type 304(4) (5) Stainless steel anchors type 316
Open to airborne salts and rain washed	
A2	Hot-dip galvanised steel(2) (6) Hot-dip galvanised anchors(2) (6)
B1, B2(3), C	Stainless steel type 304 (4),(5) Stainless steel anchors type 316

NOTE –
(1) Anchors include cast-in, mechanical or chemical anchors.
(2) All galvanising weights to steel are to comply with Table 3.10
(3) Material/protection requirements apply only to the B2 zone where not permanently submerged in seawater. Material/protection requirements, where metallic fixings or fastenings are permanently submerged in seawater, should be the subject of a special study.
(4) Type 304 stainless steel may have surface rust. Type 316 should be used where appearance is a consideration.
(5) Where there is a final cover of at least 30 mm to any part of the fixing, galvanised fixings may be substituted.
(6) The minimum requirement is hot-dip galvanised steel plus additional protection, such as epoxy powder coating, under conditions defined in 4.4.4 and 4.4.5 in NZS 3604. Type 304 stainless steel is also a suitable option.

3.13.2 *Galvanised fixings*

Galvanised steel components shall have galvanised coating masses to meet a 50-year durability in accordance with Table 3.10.

Table 3.10 – Galvanising of steel components

Component	Standard	Material protection
Bolts in any location that require galvanising (refer Table 3.9)	AS 1214	375 g/m^2 average
Fixing plates, angles	AS/NZS 2699.3 or AS/NZS 4680	600 g/m^2

3.14 Restrictions on chemical content in concrete

3.14.1 *Restriction on chloride ion content for corrosion protection*

3.14.1.1 *Added chloride*

Chloride salts or chemical admixtures formulated with greater than 0.1 % by weight of chloride shall not be added to any steel reinforced concrete required for exposure classifications B1, B2 or C, or to any prestressed or steam cured concrete.

3.14.1.2 *Total chloride*

The calculated or tested total chloride content of all steel reinforced concrete based on measurements of chloride content arising from aggregate, mixing water (including slurry water) and admixtures shall not exceed the values given in Table 3.11.

Table 3.11 – Maximum values of chloride ion content in concrete as placed

Type of member	Maximum acid soluble chloride ion content (kg/m^3 of concrete)
Prestressed concrete	0.50
Reinforced concrete exposed to moisture or chloride in service	0.80
Reinforced concrete that will be dry or protected from moisture in service	1.6

3.14.1.3 *Testing for chloride content*

When testing is performed to determine the acid soluble chloride ion content, test procedures shall conform either to ASTM C1152, AS 1012.20, or alternatively using XRF from a suitably experienced IANZ accredited chemical laboratory.

3.14.2 *Restriction on sulphate content*

The sulphate content of concrete as placed, expressed as the percentage by mass of acid soluble SO_3 to cement shall not be greater than 5.0 %.

3.14.3 *Restriction on other salts*

Other salts shall not be added to concrete unless it can be shown that they do not adversely affect durability.

3.15 Alkali silica reaction

In some parts of New Zealand where concrete aggregates are potentially reactive with alkalis, precautions may need to be taken to minimize the risk of structural damage from alkali silica reaction. CCANZ TR 3 'Alkali Silica Reaction' gives details of these aggregate types and specification precautions.

4 DESIGN FOR FIRE RESISTANCE

4.1 Notation

A_c area of concrete, mm^2
A_s area of reinforcement, mm^2
a axis distance, mm
b column width, beam width, or wall thickness, mm
b_w web thickness, mm
H_c connected height, mm
L_y longer span of a two-way slab, mm
L_x shorter span of a two-way slab, mm
N_f^* factored design load on a column, N
N_u axial load capacity of a column at normal temperature, N
t wall thickness, mm
η_{fi} the ratio of factored design load in fire resistance to axial load capacity at normal temperature

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4.2 Scope

The provisions of this section set out the requirements for the design of reinforced and prestressed concrete structures and members to resist the effects of fire, and gives methods for determining the fire resistance ratings required by the New Zealand Building Code.

4.3 Design performance criteria

4.3.1 *General performance criteria*

4.3.1.1 *Required fire resistance*

A member shall be designed to have a fire resistance rating (FRR) for each of structural adequacy, integrity and insulation equal to or greater than the required fire resistance.

4.3.1.2 *Integrity*

The criteria for integrity shall be considered to be satisfied if the member meets the criteria for both insulation and structural adequacy for that period, if applicable.

4.3.1.3 *Shear, torsion and anchorage*

Unless stated otherwise within this section when using the tabulated data or charts no further checks are required concerning shear and torsional capacity or anchorage details.

4.3.1.4 *Use of tabulated data or calculation*

The fire resistance rating (FRR) of concrete elements may be assessed using the tabulated data given in 4.4 to 4.7, or by calculation as specified in 4.10.

4.3.2 *General rules for the interpretation of tabular data and charts*

Linear interpolation between values given in the tables and charts is permitted. Values in the tables provide minimum dimensions for fire resistance. Some values of the axis distance of the reinforcement or tendons will result in covers less than those required for durability or compaction and are provided only to allow interpolation within the table or chart.

4.3.3 *Increase in axis distance for prestressing tendons*

The required axis distance for reinforcing bars shown in the tables shall be increased by the following distances where prestressing tendons are used:

(a) For prestressing bars ..... 10 mm; and
(b) For prestressing strand and wires ..... 15 mm.

4.3.4 *Joints*

Joints between members or between adjoining parts shall be constructed so that the fire resistance of the whole assembly is equal to or greater than that required for the member. The adequacy of methods used to protect service penetrations and control joints in walls or slabs shall be determined by testing in accordance with AS 1530: Part 4. Additional guidance can be found in AS 4072:Part 1.

4.3.5 *The effect of chases*

In concrete members subject to fire, chases shall be kept to a minimum. The effect of chases on the FRRs of walls shall be taken into account in accordance with the provisions of 4.7.3. The effects of chases in other members shall be taken into account using rational methods of analysis.

4.3.6 *Increasing FRRs by the addition of insulating materials*

4.3.6.1 *Use of insulation*

The FRRs for insulation and structural adequacy of a concrete member may be increased, by the addition to the surface of an insulating material, to provide increased thickness to the member, or greater insulation to the longitudinal reinforcement or tendons, or both in accordance with the provisions of 4.9.

4.3.6.2 *Slabs*

For slabs, the FRRs may be increased by the addition of toppings and/or the application of insulating materials to the soffit.

4.3.6.3 *Other methods*

For walls, the FRRs may be increased, in accordance with 4.9, by the application of insulating materials to the face exposed to fire.

4.3.6.4 *Use of other methods*

In either case, other methods (e.g. addition of insulation materials in hollow-cores) may be used. Any increase afforded shall be determined in accordance with 4.9.

4.4 Fire resistance ratings for beams

4.4.1 *Structural adequacy for beams incorporated in roof or floor systems*

A2 The fire resistance period for structural adequacy for a beam incorporated in a roof or floor system, is given by:

(a) For simply supported beam Table 4.1; or

(b) For continuous beams Table 4.2; provided the beam:

  (i) Has the upper surface integral with, or protected by, a slab complying with clause 4.5;

  (ii) Has a web of uniform width or one which tapers uniformly over its depth;

  (iii) Is proportioned so that:

    (A) The beam width measured at the centroid of the lowest level of longitudinal bottom reinforcement; and

    (B) The axis distance to the longitudinal bottom reinforcement are not less than the values given in the appropriate tables;

    (C) The web width is not less than the values given in the appropriate table.

For the purpose of this clause, a beam shall be considered continuous if, under imposed action, it is designed as flexurally continuous at one or both ends.

Table 4.1 – Fire resistance criteria for structural adequacy for simply-supported beams

Fire resistance rating (minutes)		Minimum dimensions (mm)				
		Possible combinations of a^* and b^*				Web thickness $b_w{}^*$ (mm)
30	b	80	120	160	200	80
	$a^\#$	25	20	15	15	
60	b	120	160	200	300	100
	$a^\#$	45	35	30	25	
90	b	150	200	300	400	100
	$a^\#$	55	45	40	35	
120	b	200	240	300	500	120
	$a^\#$	65	60	55	50	
180	b	240	300	400	600	140
	$a^\#$	80	70	65	60	
240	b	280	350	500	700	160
	$a^\#$	90	80	75	70	
Column 1	2	3	4	5	6	7

LEGEND:

* Where a is the axis distance
b is the minimum width of the beam
b_w is the minimum width of the web (for a non-rectangular section) | A2

\# In beams with only one layer of bottom reinforcement the distance from the centreline of the corner bar to the side of the beam (or tendons or wires) shall be increased by 10 mm except where the value of b is greater than that given in column 5 no increase is required. | A2

NOTE – For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

Table 4.2 – Fire resistance criteria for structural adequacy for continuous beams

Fire resistance rating (minutes)		Minimum dimensions (mm)			
		Possible combinations of a^* and b^*			Web thickness $b_w{}^*$ (mm)
30	b	80	160	200	80
	$a^\#$	15	12	12	
60	b	120	200	300	100
	$a^\#$	25	12	12	
90	b	150	250	400	100
	$a^\#$	35	25	25	
120	b	200	300	500	120
	$a^\#$	45	35	30	
180	b	240	400	600	140
	$a^\#$	60	50	40	
240	b	280	500	700	160
	$a^\#$	75	60	50	
Column 1	2	3	4	5	6

LEGEND:

* Where a is the axis distance
b is the minimum width of the beam
b_w is the minimum width of the web (for a non-rectangular section) | A2

\# In beams with only one layer of bottom reinforcement the distance from the centreline of the corner bar to the side of the beam (or tendons or wires) shall be increased by 10 mm except where the value of b is greater than that given in column 5 no increase is required. | A2

NOTE – For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

4.4.2 *Structural adequacy for beams exposed to fire on all sides*

A beam of approximately rectangular cross section, which can be exposed to fire on all four sides, has a particular fire resistance rating for structural adequacy if it is proportioned so that:

(a) The total depth of the beam is equal to or greater than the least value of *b*, obtained from Table 4.1 or Table 4.2 as appropriate;

(b) The cross-sectional area of the beam is equal to or greater than twice the area of a square with a side equal to *b* determined as for Item (a); and

(c) The axis distance is equal to or greater than the value determined using the minimum dimension of the beam for *b* in the relevant Table and applies to all longitudinal reinforcement or tendons.

4.5 Fire resistance ratings for slabs

4.5.1 *Insulation for slabs*

A slab has one of the FRRs for insulation given in Table 4.3 if the effective thickness of the slab is equal to or greater than the corresponding value given in the table.

The effective thickness of the slab to be used in Table 4.3 shall be taken as follows:

(a) For solid slabs, the actual thickness;

(b) For hollow-core slabs, the net cross-sectional area divided by the width of the cross section;

(c) For ribbed slabs, the thickness of the solid slab between the webs of adjacent ribs.

Table 4.3 – Fire resistance criteria for insulation for slabs

FRR for insulation (minutes)	**Effective thickness** (mm)
30	60
60	75
90	95
120	110
180	140
240	165

4.5.2 *Structural adequacy for slabs*

A slab has one of the FRRs for structural adequacy if it is proportioned so that:

(a) For solid or hollow-core slabs if, for the appropriate support conditions, the axis distance to the bottom layer of reinforcement and tendons is equal to or greater than the corresponding value given in Table 4.4.

(b) For flat slabs the axis distance to the bottom layer of reinforcement and tendons is equal to or greater than the corresponding value given in Table 4.5, provided that:

  (i) The moment redistribution used in the analysis does not exceed 15 %; and

  (ii) At least 20 % of the total top reinforcement in each direction over intermediate supports shall be continuous over the full span and placed in the column strip.

(c) For ribbed slabs if, for the appropriate support conditions it is proportioned so that:

A2 |

  (i) The width of the ribs and the axis distance to the longitudinal bottom reinforcement in the ribs are equal to or greater than those given in Table 4.6; and

  (ii) The axis distance to the bottom reinforcement in the slab between the ribs is equal to or greater than that determined in accordance with Item (a) above.

For the purpose of this clause, a slab shall be considered continuous if, under imposed load, it is flexurally continuous at least at one end.

Table 4.4 – Fire resistance ratings for solid and hollow-core slabs

Fire resistance rating (minutes)	Axis distance, *a*, to bottom layer of reinforcement [1] (mm)			
	Simply supported slabs			Continuous slabs (one-way and two-way)
	One-way	Two-way [2]		
		L_y/L_x [3] ≤ 1.5	1.5 < L_y/L_x [3] ≤ 2	
30	10	10	10	10
60	20	10	15	10
90	30	15	20	15
120	40	20	25	20
180	55	30	40	30
240	65	40	50	40

NOTE –

(1) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

(2) The axis distance for simply-supported two-way slabs applies only if the slabs are supported at all four edges. In other cases the slab shall be treated as a one-way slab.

(3) Where L_y is the longer span of a two-way slab
L_x is the shorter span of a two-way slab

Table 4.5 – Fire resistance ratings for flat slabs

Fire resistance rating (minutes)	Minimum dimensions (mm)	
	Slab thickness	Axis distance
30	150	10
60	180	15
90	200	25
120	200	35
180	200	45
240	200	50

NOTE –

(1) The axis distance relates to the reinforcement in the lower layer.

(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

Table 4.6 – Fire resistance criteria for structural adequacy for ribbed slabs

Fire resistance rating (minutes)	Simply supported one-way and two-way ribbed slabs		Continuous one-way and two-way ribbed slabs	
	Minimum width of rib (mm)	Axis distance (mm)	Minimum width of rib (mm)	Axis distance (mm)
30	80	15	80	10
60	100	35	100	25
	120	25	120	15
	≥200	15	≥200	10
90	120	45	120	35
	160	40	160	25
	≥250	30	≥250	15
120	160	60	160	45
	190	55	190	40
	≥300	40	≥300	30
180	220	75	310	60
	260	70	600	50
	≥410	60		
240	280	90	450	70
	350	75	700	60
	500	70		

NOTE –
(1) The axis distance is measured to the longitudinal bottom reinforcement.
(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.
(3) The minimum slab thickness in the flange should comply with Table 4.3 but be equal to or greater than 75 mm.

4.6 Fire resistance ratings for columns

4.6.1 *Insulation and integrity for columns*

FRRs for insulation and integrity are required for columns only where columns form part of a wall required to have a separating function. In this situation the column shall comply with the appropriate criteria for walls given in 4.7.1.

4.6.2 *Structural adequacy for columns*

The FRR for square, rectangular or circular columns shall be determined by using the minimum dimensions shown in Table 4.7.

The value of the load level, η_{fi}, shall be taken as 0.7 or calculated as follows:

$$\eta_{fi} = \frac{N_f^*}{N_u} \quad \text{.....(Eq. 4–1)}$$

where
N_f^* is the factored design axial load on the column in fire conditions
N_u is the axial load capacity of the column at normal temperature

Where $A_s \geq 0.02A_c$ and the required FRR is greater than 90 minutes, the bars shall be distributed along the sides of the column.

The dimension b in Table 4.7 for columns exposed on one side only applies to columns that lie flush with a wall having the same FRR, or to columns protruding from the wall providing that the part within the wall is able to carry the whole load. Openings in the wall shall not be nearer to the column than the minimum dimension b for the column for the FRR. Otherwise the column shall be treated as a column exposed on more than one side.

Table 4.7 – Fire resistance criteria for structural adequacy for columns

Fire resistance rating (minutes)		Minimum dimensions (mm)			
		Column exposed on more than one side			Column exposed on one side
		$\eta_{fi} = 0.2$	$\eta_{fi} = 0.5$	$\eta_{fi} = 0.7$	$\eta_{fi} = 0.7$
30	b	200	200	200	155
	a	25	25	30	25
60	b	200	200	250	155
	a	25	35	45	25
90	b	200	300	350	155
	a	30	45	50	25
120	b	250	350	350	175
	a	40	45	55	35
180	b	350	350	450	230
	a	45	60	70	55
240	b	350	450	500	295
	a	60	75	70	70
Column 1	2	3	4	5	6

NOTE –
(1) See 4.6.2
(2) For prestressing tendons, the increase in axis distance given in 4.3.3 shall be noted.

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4.7 Fire resistance ratings for walls

4.7.1 *Insulation for walls*

A wall has the fire resistance rating for insulation given in Table 4.8 if the effective thickness of the wall is equal to or greater than the corresponding value given in the table.

The effective thickness of the wall to be used in Table 4.8 shall be taken as follows:

(a) For solid walls, the actual thickness;

(b) For hollow-core walls the net cross-sectional area divided by the length of the cross section.

Table 4.8 – Minimum effective thickness for insulation

Fire resistance rating (minutes)	Effective thickness (mm)
30	60
60	75
90	95
120	110
180	140
240	165

4.7.2 *Structural adequacy for walls*

The FRR for structural adequacy for a wall shall be determined by using the values for minimum dimensions shown in Table 4.9.

Table 4.9 – Fire resistance criteria for structural adequacy for load-bearing walls

Fire resistance rating (minutes)		Minimum dimensions (mm)	
		Wall exposed to fire on one side	
		$\eta_{fi} = 0.35$	$\eta_{fi} = 0.7$
30	*b*	100	120
	a	10	10
60	*b*	110	130
	a	10	10
90	*b*	120	140
	a	20	25
120	*b*	150	160
	a	25	35
180	*b*	180	210
	a	40	50
240	*b*	230	270
	a	55	60
Column 1	2	3	4

NOTE –
(1) $\eta_{fi} = N_f^*/N_u$ see 4.6.2.
(2) For prestressing tendons the increase in axis distance given in 4.3.3 shall be noted.

4.7.3 *Chases and recesses for services in walls*

4.7.3.1 *When the effect of chases and recesses may be ignored*

The effect of chases and recesses for services, on the fire resistance ratings for structural adequacy, integrity and insulation of a wall, shall be ignored if the thickness of wall remaining under the bottom of the chase or recess is equal to or greater than half the wall thickness and the total recessed area, within any 5 m^2 of wall face, is not more than 10,000 mm^2 on one or both faces of the wall.

4.7.3.2 *Taking account of chases and recesses*

If the above limits are exceeded, the wall thickness, t, used to determine fire resistance ratings shall be taken as the overall thickness less the depth of the deepest chase or recess.

4.8 External walls that could collapse outwards in fire

4.8.1 *Application*

This clause applies to external walls which could collapse outwards from a building as a result of internal fire exposure. All such walls shall:

(a) Be attached to the building structure by steel connections;
(b) Be restrained by these connections, when subject to fire, from outward movement of the wall relative to the building structure; and
(c) Comply with the appropriate provisions of this Standard for walls.

4.8.2 *Forces on connections*

During fire exposure, the connections between each wall and the supporting structure shall be designed to resist all anticipated forces. In the absence of a detailed analysis, the connections shall be designed to resist the largest of:

(a) For all walls, the force resulting from a face load of 0.5 kPa;
A2 (b) For walls fixed to a flexible structure of unprotected steel, the force required to develop the nominal flexural strength of the wall at its base;
(c) For walls fixed to a rigid structure such as reinforced concrete columns or protected steel columns or another wall at right angles, the force required to develop the nominal flexural strength of the wall at mid-height.

4.8.3 *Design of connections*

To allow for reduced capacity in fire conditions, the fixings in the wall shall be designed as follows:

(a) Components made from unprotected mild steel shall be designed using 30 % of the yield strength of the steel in ambient conditions;

(b) Components made from other types of steel shall be designed using the mechanical properties of the steel at 680 °C;

(c) Proprietary inserts shall be designed for a minimum fire resistance rating of at least 60 minutes for unsprinklered buildings and 30 minutes for sprinklered buildings.

4.8.4 *Fixing inserts*

The fixings of inserts shall comply with the following:

(a) Proprietary cast-in or drilled-in inserts with an approved fire resistance rating shall be installed in accordance with the manufacturer's specifications;

(b) Cast-in inserts without an approved fire resistance rating shall be anchored into the wall by steel reinforcement or fixed to the wall reinforcement;

(c) Adhesive anchors shall only be used if they have a fire resistance rating tested in accordance with ISO 834:Part 1, DIN 4102-2 or AS 1530:Part 4, and are used in accordance with the manufacturer's specifications. A2

4.8.5 *Walls spanning vertically*

Walls that span vertically shall have at least two upper connections per wall panel except where several narrow panels are connected to each other to act as a single unit in which case there shall be at least two upper connections per single unit. A2

4.8.6 *Walls spanning horizontally*

Walls that span horizontally between columns shall have at least two connections per column.

4.9 Increase of fire resistance periods by use of insulating materials

4.9.1 *General*

The fire resistance ratings for insulation and structural adequacy of a concrete member may be increased by the addition to the surface of an insulating material to provide increased thickness to the member, or greater insulation to the longitudinal reinforcement or tendons, or both.

4.9.2 *Acceptable forms of insulation*

Acceptable forms of insulation include the following:

(a) Thicknesses of 1:4 vermiculite concrete or of 1:4 perlite concrete, which are appropriately bonded to the concrete;

(b) Gypsum-vermiculite plaster or gypsum-perlite plaster, both mixed in the proportion of 0.16 m^3 of aggregate to 100 kg of gypsum, in the form of either thickness added and appropriately bonded to the concrete, or as a sprayed or trowelled application applied *in situ*;

(c) Any other fire protective building board or material, that has been demonstrated to be suitable for the purpose in a standard fire resistance test.

4.9.3 *Thickness of insulating material*

4.9.3.1 *Thickness determined by testing*

The minimum thickness of insulating material added to attain the required fire resistance rating shall be determined by testing in accordance with AS 1530:Part 4.

4.9.3.2 *Thickness determination in absence of testing*

In the absence of such testing and only for the materials specified in 4.9.2, the minimum thickness of insulating material to be added may be taken as the difference between the required axis distance or effective thickness specified in this section and the actual axis distance or effective thickness, whichever governs, multiplied by: A2

(a) 0.75, for materials specified in 4.9.2 (a) and (b); or

(b) An appropriate factor for materials specified in 4.9.2(c), where the factor is derived from tests in which the difference calculated above lies within the range of insulation thicknesses tested.

4.9.4 *Reinforcement in sprayed or trowelled insulating materials*

Where the thickness of sprayed or trowelled insulating materials exceeds 10 mm, that material shall be reinforced to prevent detachment during exposure to fire.

4.10 Fire resistance rating by calculation

The fire resistance rating of a member may be assessed by a recognised method of calculation, such as given in Eurocode 2, using the load combinations given in AS/NZS 1170:Part 0 and the ϕ factor given by 2.3.2.2.

5 DESIGN PROPERTIES OF MATERIALS

5.1 Notation

E_c modulus of elasticity of concrete, MPa
E_s modulus of elasticity for non-prestressed reinforcing steel, MPa
E_{sp} modulus of elasticity of tendons, MPa
f'_c specified compressive strength of concrete, MPa
f_{ct} indirect tensile strength as defined in AS 1012:Part 10, MPa A2
f_{pu} ultimate tensile strength of prestressing steel, MPa
f_{py} yield strength of tendons, MPa
f_r average modulus of rupture, MPa
f_y lower characteristic yield strength of longitudinal (main) reinforcement, MPa
f_{yt} lower characteristic yield strength of transverse (stirrup) reinforcement, MPa
λ factor for determining the average modulus of rupture for lightweight concrete
ν Poisson's ratio for concrete
ρ density of concrete, kg/m^3

5.2 Properties of concrete

Unless otherwise specified, the values given in this section shall apply for self-compacting concrete and for conventionally placed concrete.

5.2.1 *Specified compressive strength*

The specified compressive strength of the concrete, f'_c shall be equal to or greater than 25 MPa, and shall not exceed 100 MPa, without special study.

For ductile elements and elements of limited ductility, the specified strength of the concrete, f'_c, shall not exceed 70 MPa, without special study.

5.2.2 *Applicable density range*

Concrete meeting the properties described in this section shall have a saturated surface-dry density, ρ, in the range 1800 kg/m^3 to 2800 kg/m^3.

5.2.3 *Modulus of elasticity*

The modulus of elasticity, E_c, for concrete shall be taken from the appropriate value given below: A2

(a) From a value established by testing of plain concrete in environmental conditions representative of those to which the proposed structure will be exposed;

(b) From the expression

$$\left[3320\sqrt{f'_c}+6900\right]\left(\frac{\rho}{2300}\right)^{1.5} \text{ MPa}$$

or for normal weight concrete

$$\left[3320\sqrt{f'_c}+6900\right] \text{ MPa;}$$

(c) In analyses in which strain induced actions are critical, the elastic modulus shall be equal to or greater than the value corresponding to a concrete strength of (f'_c + 10) MPa;

(d) In analyses of members for the serviceability limit state, the value of E_c given in (c) may be used in lieu of the value in (b).

5.2.4 *Modulus of rupture*

The average modulus of rupture, f_r, which may be used for calculating the moment sustained at flexural cracking for the purpose of calculation deflections shall be calculated from (a) or (b) as appropriate:

A2 (a) The average modulus of rupture for concrete, f_r, used for serviceability calculations shall be taken as $0.6\lambda\sqrt{f_c'}$

where

λ = 0.85 for normal weight sand, lightweight coarse aggregate concrete
λ = 0.75 for lightweight sand, lightweight coarse aggregate concrete
λ = 1.0 for concrete with no lightweight aggregates.

(b) When the indirect tensile strength of concrete, f_{ct}, is specified and lightweight concrete is used, f_r may be taken as:

$f_r = 1.12\ f_{ct}$ .....(Eq. 5–1)

but not more than $0.6\lambda\sqrt{f_c'}$ (MPa).

5.2.5 *Modulus of rupture from testing*

The average modulus of rupture f_r of concrete may be determined statistically from:
(a) Modulus of rupture tests carried out in accordance with AS 1012:Part 11; or
(b) Indirect tensile strength tests carried out in accordance with AS 1012:Part 10.

Where the flexural strength, f_r, is based on modulus of rupture or on indirect tensile strength tests, allowance shall be made for the decrease in flexural strength with increase in size.

A2 ### 5.2.6 *Direct tensile strength concrete*

The design direct tensile strength of normal density concrete, in the absence of more accurate data, may be taken as $0.36\sqrt{f_c'}$, or 0.54 times the indirect tensile strength obtained from the Brazil test, as given in AS 1012:Part 10.

5.2.7 *Poisson's ratio*

Poisson's ratio for normal density concrete, ν, shall be taken as 0.2 or as determined from suitable test data.
For lightweight concrete, ν, shall be determined from tests.

5.2.8 *Stress-strain curves*

A stress-strain curve for concrete may be either:
(a) Assumed to be of curvilinear form defined by recognised simplified equations; or
(b) Determined from suitable test data.

5.2.9 *Coefficient of thermal expansion*

For concrete of an aggregate type listed in Table 5.1, the coefficient of thermal expansion shall be taken as listed in the table. The coefficient of thermal expansion may be taken as 12 x 10^{-6}/°C or determined from suitable test data for other aggregate types.

Table 5.1 – Design values of coefficient of thermal expansion for concrete

Aggregate	Greywacke	Phonolite	Basalt	Andesite
Coefficient of thermal expansion x10^{-6}/°C	9.5 - 11.0	10.0 - 11.0	9.0 - 10.0	7.0 - 9.0

For self-compacting concrete these values shall be increased by 15 %.

5.2.10 *Shrinkage*

The design unrestrained shrinkage strain may be determined by testing to AS 1012 Part 13, or from appropriate published values. Appropriate allowance shall be made for the duration of measurement of shrinkage and influence of the size of member on shrinkage.

5.2.11 *Creep*

The creep coefficient used for design may be determined by testing to AS 1012 Part 16, or to ASTM C512, or assessed from appropriate published values. Appropriate allowance shall be made for the duration of measurement of creep and influence of the size of member on creep.

5.3 Properties of reinforcement

5.3.1 *Use of plain and deformed reinforcement*

All reinforcement other than ties, stirrups, spirals, shear studs, seven-wire strand, welded wire fabric and wire, strands and high strength alloy steel bars for prestressing tendons shall be deformed unless there is special reason for using plain bars.

5.3.2 *Reinforcement grades*

5.3.2.1 *Reinforcement to comply with AS/NZS 4671*

Reinforcing bars shall conform to AS/NZS 4671. Grade 500 reinforcement shall be manufactured using either the microalloy process or the in-line quenched and tempered process. However, where the in-line quenched and tempered process, or equivalent, is used the restrictions of 5.3.2.2 shall apply.

5.3.2.2 *Restrictions on in-line quenched and tempered reinforcement*

Reinforcement bars manufactured by the in-line quenched and tempered process shall not be used where welding, hot bending, or threading of bars occurs. A2

5.3.2.3 *Ductility class*

Reinforcement bars shall be ductility Class E unless the conditions of 5.3.2.4 for the use of Class N are satisfied. Ductility Class L reinforcement bars shall not be used.

5.3.2.4 *Restrictions on use of Class N reinforcement*

Class N reinforcement may be used only where either condition (a) or (b) is satisfied.

(a) Where a member is not subjected to seismic actions and the strain sustained at the ultimate limit state does not exceed 0.033 when allowance is made for:
   (i) Strains associated with stage by stage construction;
   (ii) Strains associated with cracking arising from heat of hydration movements, differential temperature effects and creep and shrinkage movements.

(b) Where a member is subjected to seismic actions but the strain in the ultimate limit state does not exceed a value of 0.025 when allowance is made for:
   (i) Strains induced in plastic hinge regions due to rotation and elongation;
   (ii) Strains in diaphragms due to deformation caused by elongation of beams/walls, and structural actions due to the transfer of forces between lateral force-resisting elements;
   (iii) Structural actions arising from dynamic magnification effects.

5.3.2.5 *Welded wire fabric*

Welded wire fabric shall be manufactured to AS/NZS 4671.

5.3.2.6 *Ductile welded wire fabric*

Welded wire fabric shall have a uniform elongation, as defined by AS/NZS 4671, of at least 10 % unless the conditions of 5.3.2.7 for the use of lower ductility welded wire fabric are satisfied.

5.3.2.7 *Lesser ductility welded wire fabric*

Lesser ductility welded fabric may be used where:

(a) The yielding of reinforcement will not occur at the ultimate limit state; or

(b) The consequences of yielding or rupture do not affect the structural integrity of the structure.

5.3.2.8 *Welding and bending of reinforcing bars*

The provisions of NZS 3109 shall apply to the welding, bending and re-bending of reinforcing bars. The method of manufacture, either microalloyed or quenched and tempered shall be taken into account.

5.3.3 *Strength*

The lower characteristic yield strength of longitudinal (main) reinforcement, f_y, used in design shall be equal to or less than 500 MPa. The lower characteristic yield strength for transverse (stirrup) reinforcement, f_{yt}, shall not be taken as greater than 500 MPa for shear or 800 MPa for confinement.

Reinforcement with a lower characteristic yield strength other than 300 MPa shall carry permanent identification.

5.3.4 *Modulus of elasticity*

The modulus of elasticity, E_s, for non-prestressed reinforcing steel shall be taken as 200,000 MPa.

5.3.5 *Coefficient of thermal expansion*

The coefficient of thermal expansion for reinforcing steel shall either be taken as 12 x 10^{-6}/°C or determined from suitable test data.

5.4 Properties of tendons

5.4.1 *Strength*

A2 | The characteristic tensile strength of tendons (f_{pu}) for commonly used tendons shall be taken as the minimum tensile strength specified in Table 5.2. For tendons of dimensions not covered, refer to AS/NZS 4672.

The yield strength of tendons (f_{py}) may either be taken as the 0.1 % or 0.2 % proof force as specified in AS/NZS 4672 or determined by test data. In the absence of test data it shall be taken as:

A2 | (a) For wire used in the as-drawn condition ..... $0.75f_{pu}$
(b) For stress-relieved wire ..... $0.85f_{pu}$
(c) For all grades of strand and bar tendons ..... $0.85f_{pu}$

Table 5.2 – Tensile strength of commonly used wire strand and bar

Tendon material type and Standard	Nominal diameter (mm)	Area (mm^2)	Minimum breaking load (kN)	Nominal tensile strength (f_{pu}) (MPa)
Stress-relieved wire AS/NZS 4672-1	5.0	19.6	32.7	1670
	5.0	19.6	34.7	1770
	7.0	38.5	64.3	1670
7-wire ordinary strand, AS/NZS 4672-1	9.3	51.6	88.8	1720
	12.4	92.9	184	1720
	12.9	100	186	1860
	15.2	140	250	1790
Hot rolled bars, AS/NZS 4672-1	26	562	579	1030
	32	840	865	1030
	36	995	1025	1030

5.4.2 *Modulus of elasticity*

The modulus of elasticity, E_{sp}, of tendons shall be either:

(a) Taken as equal to:
  (i) For stress-relieved wire to AS/NZS 4672..... 200 x 10^3 MPa
  (ii) For stress-relieved steel strand to AS/NZS 4672..... 195 x 10^3 MPa
  (iii) For hot rolled steel bars to AS/NZS 4672 ..... 200 x 10^3 MPa; or
(b) Determined by test.

Consideration shall be given to the fact that the modulus of elasticity of tendons may vary by ±5 %.

5.4.3 *Stress-strain curves*

A stress-strain curve for tendons may be determined from appropriate test data.

5.4.4 *Relaxation of tendons*

Relaxation of tendons is covered in 19.3.4.3.4.

5.5 Properties of steel fibre reinforced concrete

The design properties of steel fibre reinforced concrete shall be determined by means of deflection controlled bending tests with the specific fibre to be used, or with this information supplied by the fibre manufacturer. The methods of Appendix A to the Commentary on Section 5 may be used.

NOTES

6 METHODS OF STRUCTURAL ANALYSIS

6.1 Notation

a_s length of a support in the direction of the span, mm
A_s' area of longitudinal reinforcement in compression zone, mm^2 A2
b width of compression face of member, mm
c neutral axis depth, mm
d distance from extreme compression fibre to centroid to tension reinforcement, mm
E_c modulus of elasticity for concrete, MPa
f_c' specified compressive strength of concrete, MPa
f_r average modulus of rupture, MPa
G dead load, N or kPa
I_{cr} moment of inertia of cracked section about the centroidal axis, mm^4
I_e effective moment of inertia, mm^4
I_g moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement, mm^4
K_{cp} factor which allows for creep and shrinkage
K_s $= \dfrac{\text{Dead load} + \text{Long term live load}}{\text{Dead load} + \text{Short term live load}}$
L length of member between centrelines of supports or span of a coupling beam, mm
L_1, L_2, L_3, L_4 Span length 1, span length 2 etc. – refer Figure C6.3
L_n clear span of member measured from face of supports, mm
L_o span length for determining static moment, mm
L_o' the smaller value of L_o for the adjoining spans, mm
L_t width of the design strip, mm
L_x clear span in short direction of rectangular slab, mm
L_y clear span in long direction of rectangular slab, mm
M_a maximum moment in member at serviceability limit state, N mm
M_{cr} cracking moment, N mm
M_o total moment for span of design strip, N mm
p proportion of flexural tension reinforcement
p' proportion of longitudinal reinforcement in compression zone, A_s'/bd A2
Q live load, N or kPa
y_t distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension, mm
β_x factor for determining moment in two-way slabs
β_y factor for determining moment in two-way slabs
ε_{pu} fracture strain of prestressing tendon
ε_{su} fracture strain of reinforcing steel

6.2 General

6.2.1 *Basis for structural analysis*

Methods of analysis for concrete structures shall take into account the following:

(a) The strength and deformational properties of the component materials;
(b) The equilibrium requirements for all forces acting on, and within, the structure;
(c) The requirements of compatibility of deformations within the structure; and
(d) The support conditions, and, where appropriate, interaction of the structure with the foundation and other connecting or adjacent structures.

6.2.2 *Interpretation of the results of analysis*

Irrespective of the method chosen for the structural analysis, the simplifications, idealisations and assumptions implied in the analysis shall be considered in relation to the real, three-dimensional nature of the structure when the results of the analysis are interpreted.

6.2.3 *Methods of analysis*

6.2.3.1 *Permissible methods*

For the purpose of complying with the requirements for stability, strength and serviceability specified in Section 2, it shall be permissible to determine the action effects and deformations in a reinforced or prestressed structure and its component members using the following methods, as appropriate:

(a) Linear elastic analysis, in accordance with 6.3;
(b) Non-linear structural analysis, in accordance with 6.4;
(c) Plastic methods of analysis for slabs and frames, in accordance with 6.5;
(d) Strut and tie method of analysis, in accordance with 6.6;
(e) Structural model tests designed and evaluated in accordance with the principles of mechanics;
(f) The following simplified methods of analysis:
    (i) For reinforced continuous beams or one-way slabs, the simplified method given in 6.7.2;
    (ii) For reinforced two-way slabs supported by walls or beams on all four sides, the simplified method given in 6.7.3;
    (iii) For reinforced two-way slab systems having multiple spans, the simplified method given in 6.7.4;
(g) For non-flexural members, the methods given in 6.6 and Section 16.

6.2.3.2 *Frames or continuous construction*

All members of frames or continuous construction shall be designed for the maximum effects of loads in the serviceability and ultimate limit states. Moments obtained from elastic analyses of structures with factored loads for the ultimate limit state may be modified according to 6.3.7. However, the redistribution of moments permitted in 6.3.7.2 or 19.3.9 shall not be applied to the approximate moments of 6.7.

6.2.3.3 *Seismic loading*

For analysis involving seismic loading, refer to 6.9 which shall take precedence over other clauses in this section.

6.2.4 *Vertical loads on continuous beams, frames and floor systems*

In the analysis of continuous beams, two-dimensional frames and floor systems, or three-dimensional framed structures and floor systems, the arrangement of vertical loads to be considered shall consist of at least the following:

(a) The factored dead load, including considerations given in 2.3.2.1.;
(b) Where the live loading pattern is fixed – the factored live load;
(c) Where the live loading pattern (Q) can vary;
    (i) For continuous beams and two-dimensional frames or floor systems:
        (A) The factored live load on alternate spans; and
        (B) The factored live load on two adjacent spans; and
        (C) The factored live load on all spans.
    (ii) For three-dimensional framed structures and floor systems, patterned variations of the factored live load on adjacent spans in a chequerboard arrangement on all spans to determine the peak design actions at each critical section.

6.3 Linear elastic analysis

6.3.1 *Application*

In the design of structures for ultimate and serviceability limit states, linear elastic analysis may be used for the purpose of evaluating internal action effects.

6.3.2 *Span lengths*

For the purposes of calculating moments, shears, deflections or stiffness, the following shall be used:

(a) In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance centre-to-centre of supports;
(b) Solid or ribbed slabs built integrally with supports, with clear spans of not more than 3 m, may be analysed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected;
(c) Span length of members not built integrally with supports shall be considered to be the clear span plus the depth of member but need not exceed the distance between centres of seatings.

6.3.3 *Analysis requirements*

The analysis shall take into account:
(a) The stress-strain curves of the steel reinforcement and tendons;
(b) Static equilibrium of the structure after redistribution of the moments; and
(c) The properties of the concrete as defined in 5.2.

6.3.4 *Critical sections for negative moments*

Circular or regular polygon shaped supports may be treated as square supports with the same area for location of the critical section for negative moment.

6.3.5 *Stiffness*

6.3.5.1 *Stiffness to be appropriate to limit state*

The stiffness of members shall be chosen to represent the conditions at the limit state being analysed.

6.3.5.2 *Variations in cross section*

The effect of haunching and other variations of cross section along the axis of a member shall be considered and, where significant, taken into account in the determination of the member stiffness.

6.3.5.3 *Assumptions to be applied appropriately*

Any assumptions regarding the relative stiffness of members shall be applied in a consistent and appropriate manner for the actions being considered.

6.3.5.4 *Effective stiffness*

Refer to 6.8 for effective stiffness to be used when calculating deflections.

6.3.6 *Secondary bending moments and shears resulting from prestress*

Secondary actions due to prestress or other self strain conditions in prestressed or partially prestressed members are considered in 19.3.8.

6.3.7 *Moment redistribution in reinforced concrete for ULS*

6.3.7.1 *General requirements*

6.3.7.1.1 *Redistribution permitted*

In design calculations for strength of statically indeterminate members, the elastically determined bending moments at any interior support may be reduced or increased by redistribution, provided an analysis is undertaken to show that there is adequate rotation capacity in critical moment regions to allow the assumed distribution of bending moments to be achieved. In this calculation the section stiffness shall be based on the section properties assuming the concrete cracks in flexure and neglecting tension stiffening unless a more exact analysis is made. The effective plastic hinge length on each side of a support or load point shall be taken not greater than d/2.

6.3.7.1.2 *Redistribution due to creep and foundation movement*

Consideration shall be given to the significant redistribution of internal actions that may occur due to relative foundation movements and the considerable demands this can place on the rotational capacity of the critical sections. Where staged construction of members or structures is used, creep redistribution of structural actions shall also be considered.

6.3.7.2 *Deemed to comply approach for reinforced members (for prestressed members refer to Section 19)*

For load combinations which do not include seismic actions, the requirements of 6.3.7.1.1 shall be deemed to be met provided all of the following requirements are satisfied:

(a) All of the longitudinal reinforcement in the member is ductility Class E;

(b) The elastic bending moment distribution before redistribution is determined in accordance with 6.3.5;

(c) Equilibrium between the internal forces and the external loads must be maintained under each appropriate combination of factored vertical and horizontal loads and forces;

(d) The design strength after redistribution provided at any section of a member shall be equal to or greater than 70 % of the moment for that section obtained from a moment envelope covering all appropriate combinations of loads obtained from the analysis of elastic structures;

(e) The moment at any section in a member obtained from the analysis of elastic structures due to a particular combination of factored loads and forces shall not be reduced by more than 30 % of the numerically largest moment given anywhere by the elastic moment envelope for that particular member, covering all combinations of ultimate limit state loads and forces;

(f) The neutral axis depth, c, of a section resisting a reduced moment due to moment redistribution equal to or less than:

A2

$$c = c_b\,(0.75 - B) \qquad \text{(Eq. 6–1)}$$

where B is the absolute value of the ratio of the reduction in moment of resistance to the largest ultimate limit state moment, determined by elastic analysis, anywhere in the member which contains the section, and c_b is the neutral axis depth corresponding to balanced conditions.

The consequences of redistribution assumed in the ultimate limit state shall be assessed for the serviceability limit state.

6.3.8 *Idealised frame method of analysis*

The idealised frame method may be used to analyse structures of reinforced concrete and prestressed concrete that can be represented as a framework of line members with a regular layout. This method may also be applied to the analysis of framed structures with a regular layout incorporating two-way slab systems. Refer to the commentary for further details on this method.

6.4 Non-linear structural analysis

6.4.1 *General*

When used to evaluate the conditions in a structure in the serviceability limit state or at the ultimate limit state, non-linear analysis shall be carried out in accordance with the requirements of 6.2.1 to 6.2.3 and 6.4.2 to 6.4.4.

6.4.2 *Non-linear material effects*

The analysis shall take into account relevant non-linear and inelastic effects in the materials, such as:

(a) Non-linear relation between stress and strain for the reinforcement, the tendon and the concrete;

(b) Cracking of the concrete;

(c) The tension stiffening effect in the concrete between adjacent tensile cracks;

(d) Creep and shrinkage of the concrete; and

(e) Relaxation of prestressing tendons.

6.4.3 *Non-linear geometric effects*

Equilibrium of the structure in the deformed condition shall be considered whenever joint displacements or lateral deflections within the length of members significantly affect the action effects or overall structural behaviour.

6.4.4 *Values of material properties*

Non-linear analysis shall be undertaken using material stress-strain relationships based on either mean material strengths or design strengths of the material. However, in all cases the ultimate limit state section strengths shall be based on design strengths.

6.5 Plastic methods of analysis

6.5.1 *General*

Where plastic methods are used in determining the ultimate limit state of structures, the reinforcement shall be arranged with due regard to the serviceability requirements of the structure.

6.5.2 *Methods for beams and frames*

Plastic methods of analysis may be used for determining the ultimate limit state of continuous beams and frames provided it is shown that the high-moment regions possess sufficient moment-rotation capacity to achieve the plastic redistribution implied in the analysis.

6.5.3 *Methods for slabs*

6.5.3.1 *Use of plastic methods of analysis*

For the ultimate limit state of one-way and two-way slabs, plastic methods of analysis based on lower bound or upper bound theory may be used provided ductility Class E reinforcement is used throughout.

6.5.3.2 *Lower bound method for slabs*

The design bending moments obtained using lower bound theory shall satisfy the requirements of equilibrium and the boundary conditions applicable to the slab.

6.5.3.3 *Upper bound method for slabs*

Upper bound or yield line analysis for ultimate limit strength of a slab shall satisfy the following requirements:

(a) The design bending moments shall be obtained from calculations based on the need for a mechanism to form over the whole or part of the slab at collapse;

(b) The mechanism that gives rise to the most severe design bending moments shall be used for the design of the slab.

6.6 Analysis using strut-and-tie models

The analysis of squat elements and regions of discontinuity, where flexural theory relevant to members is not appropriate, may be based on strut-and-tie models. Requirements of equilibrium and strain compatibility shall be satisfied. Appendix A provides strut and tie methodologies.

6.7 Simplified methods of flexural analysis

6.7.1 *General*

In lieu of more detailed structural analysis, it is permissible to design ductile reinforced concrete beams and slabs for strength in accordance with the provisions of 6.7.2, 6.7.3, or 6.7.4 as appropriate. The simplified methods in this clause apply only to reinforced concrete beams or slabs containing ductility Class E steel as the principal longitudinal reinforcement.

6.7.2 *Simplified method for reinforced continuous beams and one-way slabs*

6.7.2.1 *Application*

6.7.2.1.1 *Conditions for use of simplified methods*

Simplified methods may be used for the calculation of design bending moments and shear forces for strength in continuous beams and one-way slabs of reinforced concrete construction, provided that:

(a) The ratio of the longer to the shorter length of any two adjacent spans does not exceed 1.2;
(b) The loads are essentially uniformly distributed;
(c) The live load (Q) does not exceed twice the dead load (G);
(d) Members are of uniform cross section;
(e) The reinforcement is arranged in accordance with the requirements of Section 9;
(f) Bending moments at supports are caused only by the action of loads applied to the beam or slab; and
(g) There is no redistribution of bending moments.

6.7.2.1.2 *Design information*

Refer to the commentary for the simplified method to determine:

(a) Negative design moment;
(b) Positive design moment;
(c) Design shear force.

6.7.3 *Simplified method for reinforced two-way slabs supported on four sides*

6.7.3.1 *Application*

6.7.3.1.1 *Determination of bending moments*

The design bending moments and shear forces for strength in reinforced two-way simply supported or continuous rectangular slabs, which are supported by walls or beams on four sides, may be determined by the simplified method provided that:

(a) The loads are essentially uniformly distributed;
(b) The reinforcement is arranged in accordance with the requirements of Section 12; and
(c) Bending moments at supports are caused only by the action of loads applied to the beam or slab.

6.7.3.1.2 *Referral to the commentary*

Refer to the commentary for the simplified method to determine:

(a) Design bending moments;
(b) Torsional moment at exterior corners;
(c) Load allocation onto supporting walls or beams.

6.7.4 *Simplified method for reinforced two-way slab systems having multiple spans*

6.7.4.1 *Conditions for the use of simplified method*

For multiple-span reinforced two-way slab systems; including solid slabs with or without drop panels, slabs incorporating ribs in two directions (waffle slabs) and beam and slab systems including thickened slab bands, bending moments and shear forces in both directions may be determined in accordance with this clause provided that the following requirements are met:

(a) There are at least two continuous spans in each direction;
(b) The support grid is rectangular, except that individual supports may be offset up to a maximum of 10 % of the span in the direction of the offset;
(c) In any portion of the slab enclosed by the centrelines of its supporting members, the ratio of the longer span to the shorter span is not greater than 2.0;
(d) In the design strips in each direction, successive span lengths do not differ by more than one-third of the longer span and in no case is an end span longer than the adjacent interior span;
(e) Lateral forces on the structure are resisted by shear walls or braced frames;
(f) Vertical loads are essentially uniformly distributed;

(g) The live load (Q) does not exceed twice the dead load (G);
(h) The reinforcement is arranged in accordance with Section 12.

6.7.4.2 *Referral to the commentary*

See commentary for the simplified method to determine:
(a) Total static moment for a span;
(b) Design moments;
(c) Transverse distribution of the design bending moment;
(d) Moment transfer for shear in flat slabs;
(e) Shear forces in beam and slab construction;
(f) Openings in slabs.

6.8 Calculation of deflection

6.8.1 *General*

Deflection calculations shall take into account the effects of cracking, tension stiffening, shrinkage, creep, and relaxation. Where appropriate, consideration shall be given to deformations that may result due to deflection of the formwork or settlement of the supporting props during construction.

Calculations shall be made to ensure that under the serviceability limit state conditions the deformations are such that they do not adversely affect the serviceability of the structure. Deflection shall be calculated as described in 6.8.2 or 6.8.3.

6.8.2 *Deflection calculation with a rational model*

Rational methods of calculation may be used to determine deflections. Such methods shall make rational allowance for cracking in the concrete, the length of time the loading acts, the basic properties of concrete including its elastic, creep and shrinkage characteristics including the influence of the maturity of the concrete when the load is applied, the duration of the curing period, and the properties of the reinforcement.

6.8.3 *Calculation of deflection by empirical model*

Short-term deflections are found, as set out in (a) below, and increased to allow for additional deformation due to creep and shrinkage as set out in (b).

(a) *Short-term deflection*

The deflection that occurs immediately on the application of the dead load and short-term live load is found by the usual methods, or formulae, which are based on elastic theory. Allowance is made for the effects of cracking and reinforcement on member stiffness as set out below.

Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be calculated with the modulus of elasticity, E_c, for concrete as specified in 5.2.3 (normal density or lightweight concrete) and with the effective moment of inertia, I_e, as follows but not greater than I_g.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \quad \text{(Eq. 6–2)}$$

where

$$M_{cr} = \frac{f_r I_g}{y_t} \quad \text{(Eq. 6–3)}$$

and f_r is as defined in 5.2.4 or 5.2.5.

For continuous spans, the effective moment of inertia shall be taken as either:

(i) the average of the values obtained from Equation 6–2 for the critical positive and negative moment sections, or

(ii) The appropriate values found from Equation 6–2 for the negative and positive moment regions of the beam.

(b) *Calculation of long-term deflection*

Unless values are obtained by a more comprehensive analysis, the additional long-term deflection for flexural members (normal density and lightweight concrete) shall be obtained by multiplying the short-term deflection by K_s and by K_{cp}. K_s is the ratio of the maximum bending moment due to dead and long-term loading divided by the corresponding moment due to dead and short-term live loading. The value of K_{cp}, which allows for the additional deflection arising from creep and shrinkage in concrete, is given by:

$$K_{cp} = \frac{2}{1+50p'} \quad \text{.....(Eq. 6–4)}$$

where p' is the proportion of longitudinal reinforcement in the compression zone. The rate at which this deflection may be expected to develop can be assessed from the creep-time curve for the appropriate concrete.

The resultant deflection is the sum of the short-term and long-term values.

6.8.4 *Calculation of deflection – prestressed concrete*

Where the deflection of prestressed members is determined:

(a) Where the member is designed not to form flexural cracks in the serviceability limit state, (tensile stress less than the limiting value for class T members in 19.3.3.5.1), the short-term deflection shall be determined by recognised elastic theory, using either section properties based on gross or transformed section properties.

(b) Where the section is designed to crack in flexure in the serviceability limit state rational allowance shall be made for the reduction in stiffness of the member, or zone where cracking is anticipated. Section stiffness shall be based on the transformed section ignoring concrete in the tension zone, or alternatively Equation 6–2 may be used in which M_{cr} is adjusted to allow for the effect of prestress on the flexural cracking moment.

(c) Additional long-term deflection for prestressed concrete members shall be calculated taking into account the stresses in the concrete and reinforcement under sustained load including the effects of creep and shrinkage of the concrete and relaxation of the prestressed reinforcement. (Refer to commentary, Appendix CE for guidance.

6.8.5 *Shored composite construction*

6.8.5.1 *Deflection after the removal of supports*

If composite flexural members are supported during construction so that, after removal of temporary supports, the dead load is resisted by the full composite section, the composite member may be considered equivalent to a monolithically cast member for calculation of deflection. Account shall be taken of the curvatures resulting from differential shrinkage of precast and cast-in-place components, and of the axial creep effects in a prestressed concrete member.

6.8.5.2 *Deflection of non-prestressed composite members*

The long-term deflection of the precast member shall be investigated including the magnitude and duration of load prior to the beginning of effective composite action.

6.9 Additional requirements for earthquake effects

6.9.1 *Linear elastic analysis*

6.9.1.1 *Analyses to be based on anticipated levels of cracking*

Analyses involving seismic forces, used for the assessment of deflections and periods of vibration of structures, and internal actions, in the elastic range shall make allowances for the anticipated levels of concrete cracking. Deflections so determined shall be used in the estimation of design actions and displacements at the ultimate limit state for ductile structures as specified in NZS 1170.5 or other referenced loading standard.

6.9.1.2 *ULS deflections to allow for post-elastic effects*

Assessment of structural deflections for the ultimate limit state involving seismic forces shall also make due allowance for post-elastic effects and reinforcement grade.

6.9.1.3 *Walls and other deep members*

In the estimation of stiffness or deformations of structural walls and other deep members, allowance shall be made for shear deformations, and deformation due to the development of bars in the anchorage zone for the wall or deep member and the deformation of foundations, where appropriate.

6.9.1.4 *Ductile dual structures*

Wherever a combination of different ductile structural systems is used, rational analysis, taking into account the relative stiffness and location of such elements, shall be employed to allocate the seismic resistance to each element at the serviceability and ultimate limit states. Diaphragms, shall be designed to ensure the design forces can be transmitted between lateral force-resisting elements in accordance with 13.4.

6.9.1.5 *Redistribution of moments and shear forces*

In ductile or limited ductile structures redistribution of moments or shear forces, derived from an elastic analysis for factored gravity loads and seismic forces at the ultimate limit state, may be made, provided:

(a) The absolute maximum moment derived for any span from elastic analysis for any combination of earthquake forces and appropriately factored gravity loading shall not be reduced by more than 30 % as a result of redistribution;

(b) The positive span moments for all design load combinations shall be modified in beams when terminal negative or positive moments are changed, to satisfy the requirements of equilibrium;

(c) Moment redistribution shall not be used where terminal beam negative moments for any load combination are based on nominal values;

(d) The requirements of 6.3.7 shall be satisfied when the strength of the structure at the ultimate limit state is governed by gravity loads and wind forces only;

(e) Redistribution of moments due to lateral seismic force only, between cantilever or coupled structural walls, with or without ductile frames, shall not reduce the maximum value of the moment derived from elastic analysis for any wall by more than 30 %.

(f) The lateral storey shear force in any storey shall not be reduced in the moment distribution process.

6.9.1.6 *Capacity design for columns*

Refer to Appendix D for a recommended capacity design procedure for columns in ductile multi-storey frames subject to earthquake forces.

NOTES

7 FLEXURAL, SHEAR AND TORSIONAL STRENGTH OF MEMBERS WITH OR WITHOUT AXIAL LOAD

7.1 Notation

a depth of equivalent rectangular stress block as defined in 7.4.2.7, mm

A_ℓ total area of longitudinal reinforcement to resist torsion, mm^2

A_o area enclosed by line connecting the centres of longitudinal bars in the corners of closed stirrups, or for box girder type sections the area enclosed by the perimeter of the centroid of transverse reinforcement resisting the torsional shear flow, mm^2 A2

A_{co} area enclosed by perimeter of section, mm^2

A_{cv} the effective shear area, mm^2

A2

A_t area of one leg of a closed stirrup resisting torsion within a distance s, mm^2

A_{vf} area of shear-friction reinforcement, mm^2

c distance from extreme compression fibre to neutral axis, mm

d distance from extreme compression fibre to centroid of tension reinforcement, mm

E_s modulus of elasticity of steel, MPa. see 5.3.4

f'_c specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yt} design yield strength of transverse reinforcement provided for shear and/or torsion, MPa A2

M^* design moment at section at the ultimate limit state, N mm

M_n nominal flexural strength of section, N mm

N^* design axial load at ultimate limit state, N

p_c perimeter of section, mm

p_o perimeter of area, A_o, mm

s centre-to-centre spacing of shear or torsional reinforcement measured in the direction parallel to the longitudinal reinforcement, mm

t_c 0.75 A_{co}/p_o equivalent tube thickness for section prior to torsional cracking, for hollow sections not greater than the actual wall thickness, mm A2

t_o 0.75 A_o/p_c, equivalent tube thickness for torsionally cracked section, for hollow sections not greater than the actual wall thickness, mm

T_n nominal torsional strength of section, N mm

T^* design torsional moment at section at the ultimate limit state, N mm

V_c nominal shear strength provided by concrete, N

v_{max} maximum permissible shear stress, MPa

V_n total nominal shear strength of section, N

v_n nominal shear stress, MPa

V_s nominal shear strength provided by the shear reinforcement, N

v_{tn} nominal shear stress due to torsion, MPa

V^* design shear force at section at the ultimate limit state, N

α_f angle between shear-friction reinforcement and shear plane

α_1 factor defined in 7.4.2.7

β_1 factor defined in 7.4.2.7

λ reduction factor for shear-friction strength

μ coefficient of friction, see 7.7.4.3

ϕ strength reduction factor, see 2.3.2.2

7.2 Scope

The provisions of this section shall apply to the design of members for flexure and shear, including torsion with or without axial loads. Members subject primarily to flexure and shear shall be designed as beams or slabs. Members subject primarily to flexure, axial load and shear shall be designed as columns or piers.

7.3 General principles

Flexural and shear strengths may be determined independently without flexure-shear interaction effects. The influence of axial load shall be included in the determination of both the flexural and shear strength.

7.4 Flexural strength of members with shear and with or without axial load

7.4.1 *Flexural strength requirement*

Design of cross sections of members subjected to flexure, shear and with or without axial loads is based on:

$M^* \leq \phi M_n$ .....(Eq. 7–1)

Where M^* is the design bending moment at the section derived from the ultimate limit state loads and forces and M_n is the nominal flexural strength of the section.

7.4.2 *General design assumptions for flexural strength*

7.4.2.1 *Strength calculations at the ultimate limit state*

The design of members for flexure with or without axial loads at the ultimate limit state shall be based on strain compatibility and equilibrium using either:

(a) The assumptions of 7.4.2.2 to 7.4.2.9 when the full cross section is considered to contribute to the strength of the member; or

(b) Complete stress-strain relationships for reinforcing and concrete including the case when after spalling of the concrete only the core of the cross section is considered to contribute to the strength of the member. In the calculations the assumptions of 7.4.2.2 to 7.4.2.9 shall be satisfied except where spalling of the unconfined concrete is assumed to occur the limiting strain in the concrete consistent with the stress strain relationship for the concrete may be used in lieu of the of value 0.003 given in 7.4.2.3.

7.4.2.2 *Strain relationship to geometry*

Strain distribution in reinforcement and concrete shall be assumed to vary linearly through the depth of the member. For deep beams a strut-and-tie model shall be used.

7.4.2.3 *Maximum concrete strain*

The maximum strain at the extreme concrete compression fibre at the development of the nominal flexural strength shall be assumed equal to 0.003.

7.4.2.4 *Steel stress-strain relationship*

The stress in reinforcement below the lower characteristic yield strength, f_y, for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, the stress in reinforcement shall be considered independent of strain and equal to f_y.

7.4.2.5 *Concrete tensile strength*

The tensile strength of concrete shall be neglected in flexural strength calculations of reinforced concrete.

7.4.2.6 *Concrete stress-strain relationship*

The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of the nominal flexural strength in substantial agreement with the results of comprehensive tests.

7.4.2.7 *Equivalent rectangular concrete stress distribution*

The requirements of 7.4.2.6 may be considered satisfied by an equivalent rectangular concrete compressive stress distribution defined by the following:

(a) Concrete stress of $\alpha_1 f_c'$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain;

(b) The distance, c, from the fibre of maximum compressive strain to the neutral axis shall be measured in a direction perpendicular to that axis;

(c) The factor α_1 shall be taken as 0.85 for concrete strengths, f_c', up to and including 55 MPa. For strengths, f_c', above 55 MPa, α_1 shall be taken as:

$$\alpha_1 = 0.85 - 0.004 (f_c' - 55) \quad \text{(Eq. 7–2)}$$

but with a minimum value of 0.75.

(d) Factor β_1 shall be taken as 0.85 for concrete strengths, f_c', up to and including 30 MPa. For strengths above 30 MPa, β_1 shall be taken as:

$$\beta_1 = 0.85 - 0.008 (f_c' - 30) \quad \text{(Eq. 7–3)}$$

but with a minimum value of 0.65.

7.4.2.8 *Balanced conditions*

Balanced strain conditions exist at a cross section of a member when tension reinforcement near the extreme tension fibre of the cross section reaches the strain corresponding to its lower characteristic yield strength, f_y, just as the concrete in compression reaches its assumed ultimate strain of 0.003.

7.4.2.9 *Compression reinforcement*

Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the flexural strength of beams and columns. Where compression reinforcement is required to satisfy strength transverse reinforcement shall be detailed as specified in 9.3.9.6, 10.3.10.5 and 10.3.10.6.

7.5 Shear strength of members

7.5.1 *General*

Design of cross sections of members shall be based on:

$$V^* \leq \phi V_n \quad \text{(Eq. 7–4)}$$

where V^* is the design shear action at the section derived from ultimate limit state and V_n is the nominal shear strength of the section.

The nominal shear strength of a section, V_n, is given by:

$$V_n = v_n A_{cv} \quad \text{(Eq. 7–5)}$$

where v_n is the nominal shear stress and A_{cv} is the effective shear area, the values for which are defined in the appropriate sections for the type of member being considered.

7.5.2 *Maximum nominal shear stress, v_{max}*

The nominal shear stress for shear added to the shear stress due to torsion, or the shear stress due to shear friction, shall be equal to or less than v_{max}, which is the smaller of $0.2 f_c'$ or 8 MPa.

For beam column joint zones the larger of the nominal shear stresses calculated from the design shear forces in either the horizontal or vertical directions shall be equal to or less than the smaller of $0.2 f_c'$ or 10 MPa.

7.5.3 *Nominal shear strength, V_n*

The total nominal shear strength of the section V_n for all cases except for shear-friction, shall be computed from:

$V_n = V_c + V_s$ .....(Eq. 7–6)

where V_c is the nominal shear strength provided by the concrete mechanisms (7.5.4) and V_s is the nominal shear strength provided by shear reinforcement (7.5.5).

The nominal shear strength corresponding to shear-friction is defined in 7.7.

7.5.4 *Nominal shear strength provided by the concrete, V_c*

When considering determining the nominal shear strength provided by the concrete, V_c:

(a) The effects of axial tension including those due to creep, shrinkage and temperature in restrained members, shall be considered, whenever applicable. The effect of inclined flexural compression in variable depth members shall be considered.

(b) For reinforced concrete beams and one-way slabs, columns and piers, walls, two-way slabs, beam column joints and prestressed concrete are given in Sections 9, 10, 11, 12, 15 and 19 respectively.

7.5.5 *Nominal shear strength provided by the shear reinforcement*

When the design shear force V^* exceeds the design shear strength provided by the concrete ϕV_c shear reinforcement shall be provided to satisfy Equations 7–4 and 7–6. The design shear strength provided by the shear reinforcement, ϕV_s, shall be computed with V_s for reinforced concrete beams and one-way slabs, columns and piers, walls, two-way slabs, beam column joint zones and prestressed concrete as given in Sections 9, 10, 11, 12, 15 and 19 respectively.

7.5.6 *Shear reinforcement details*

Shear reinforcement may consist of:

(a) Stirrups perpendicular to the longitudinal axis of member;

(b) Stirrups making an angle of 45° or more with the longitudinal tension bars;

(c) Vertical or inclined prestressing;

(d) Mechanically anchored bars with end bearing plates having an area at least 10 times the cross-sectional area of the bar;

(e) Longitudinal reinforcement with a bent portion making an angle of 30° or more with the longitudinal tension reinforcement;

(f) Combinations of stirrups and bent longitudinal reinforcement;

(g) Spirals;

(h) Diagonally reinforced members (as in diagonally reinforced coupling beams);

(i) Welded wire mesh, in members not located in potential plastic regions;

(j) Fibres, designed to the requirements of Appendix A of the Commentary to Section 5, in members not located in potential plastic regions.

7.5.7 *Location and anchorage of reinforcement*

7.5.7.1 *Anchoring of stirrups and ties*

Stirrups, ties or wires shall enclose the flexural tension reinforcement and be anchored as close as possible to the extreme compression fibre. Such stirrups and ties shall be anchored around longitudinal reinforcement by at least a 135° stirrup hook. Alternatively stirrups shall be spliced by welding to develop the breaking strength of the bar or anchored by mechanical anchors.

7.5.7.2 *Bent up bars*

Bent up bars, which are used as shear reinforcement, shall extend from the point where the bend starts in the bar in both the tension and compression zones for a distance equal to or greater than the development length.

7.5.7.3 *Lapped splices*

Lapped splices in stirrups and ties shall not be used unless the requirements of 8.7.2.8 are satisfied. A2

7.5.8 ***Design yield strength of shear reinforcement***

Design yield strength of transverse reinforcement for torsion and or shear, f_{yt}, shall not exceed 500 MPa. A2

7.5.9 ***Alternative methods for determining shear strength***

7.5.9.1 *Equilibrium and strain compatibility methods*

In lieu of the methods specified in 7.5.4 and 7.5.5 the resistance of a member in shear, or shear combined with torsion, may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains and by using appropriate stress-strain relationships for reinforcement and for diagonally cracked concrete. A2

7.5.9.2 *Strut and tie*

Strut and tie models may be used to design for shear and/or torsion. Where this approach is used V_c shall be taken as zero. A2

7.5.10 ***Minimum area of shear reinforcement***

A minimum area of shear reinforcement is required in most members. These minimum areas are specified in 9.3.9.4.13 for beams and one-way slabs, in 10.3.10.4.4 for columns and 11.3.10.3.8(b) for walls.

7.6 Torsional strength of members with flexure and shear with and without axial loads

7.6.1 ***Members loaded in torsion***

7.6.1.1 *Exceptions*

The provisions of 7.6 shall not apply to one-way slabs or slabs complying with Section 12. A2

7.6.1.2 *Requirement for torsional reinforcement*

If the torsion in a member is required to maintain equilibrium in the structure and if the magnitude of the torsion design action required (T^*) exceeds $0.1\phi A_{co} t_c \sqrt{f_c'}$, torsion reinforcement designed in accordance with 7.6.4 shall be provided. The value of t_c shall not exceed the actual wall thickness in hollow sections. A2

7.6.1.3 *Torsion due to deformation compatibility*

If torsion in a member arises because the member twists to maintain compatibility, the effect of torsion on the member may be neglected provided the requirements of either (a) or (b) given below are satisfied: A2

(a) The torsional reinforcement is not required if the torsional design action, T^*, calculated from an analysis based on gross section properties, is equal to or less than $0.1\phi A_{co} t_c \sqrt{f_c'}$;

(b) The effect of torsion on a member may be neglected provided the moments and shears in the structure are computed assuming the member has no torsional stiffness, and the following provisions are satisfied:

  (i) In those regions of adjoining members, where bending moments may be induced by torsional restraint in the member, adequately anchored flexural reinforcement shall be added to control potential flexural cracking in the serviceability limit state;

  (ii) Torsional reinforcement shall be provided in the member in accordance with the provisions of 7.6.2 and detailed in accordance with 7.6.3.

7.6.1.4 *Sections within d of support*

Sections located less than a distance, d, from the face of the support shall be designed for the same torsion as that computed at a distance d.

7.6.1.5 *Torsional strength requirements*

Design of cross sections subject to torsion shall be based on the relationship:

$T^* \le \phi T_n$ .....(Eq. 7–7)

where T^* is the torsion at the section derived from the load on the structure at the ultimate limit state and T_n is the nominal torsional strength of the section.

7.6.1.6 *Torsional shear stress*

The torsional shear strength, v_{tn}, shall be computed from:

$$v_{tn} = \frac{T_n}{2A_o t_o}$$ .....(Eq. 7–8)

where the value of t_o shall not exceed the actual wall thickness of hollow sections and v_{tn} shall not exceed that given by 7.5.2.

7.6.1.7 *Torsion in flanged sections*

Where torsional shear stress and torsional reinforcement is determined for members with flanged sections, the value of A_o and A_{co} shall be based either on the stem of the section only, without flanges, or on the stem with flanges where the width of overhanging flange used shall not exceed three times the thickness of the flange.

7.6.1.8 *Torsional and flexural shear together*

Where torsional and flexural shear stresses occur together at a section the following condition shall be satisfied:

$v_n + v_{tn} < v_{max}$ .....(Eq. 7–9)

where v_{max} is given by 7.5.2.

A2

7.6.2 ***Reinforcement for compatibility torsion***

7.6.2.1 *Minimum reinforcement for compatibility torsion*

Where required by 7.6.1.3(b), closed stirrup and longitudinal reinforcement meeting the requirements of 7.6.3 shall be provided for a minimum nominal torsional moment, T_n, equal to or greater than the smaller of:

(a) T^*/ϕ calculated neglecting the reduction in torsional stiffness due to torsional cracking, or

(b) $T_{n,min}$, given by:

$$T_{n,min} = 0.44\, A_{co}\, t_c \sqrt{f_c'}\left(1 + \frac{N^*}{0.33\, A_g \sqrt{f_c'}}\right)$$ .....(Eq. 7–10)

where, N^* is the design axial action, taken as positive for compression, A_g is the gross section area, and for hollow sections and t_c shall not exceed the actual wall thickness.

Note - The cross-sectional area of a leg or legs of closed stirrups on one side of the member, A_t, within a spacing of s, along the member, together with the corresponding area of longitudinal reinforcement, A_ℓ located around the perimeter, p_o, shall satisfy Equation Eq. 7–11(a):

$$\left(\frac{A_t f_{yt}}{s}\frac{A_\ell f_y}{p_o}\right)^{0.5} \ge \frac{T_n}{2A_o}$$ .....(Eq. 7–11(a))

Satisfying the limits given in Equation 7–11(b):

$$\frac{0.6 A_\ell f_y}{p_o} \le \frac{A_t f_{yt}}{s} \le \frac{1.7 A_\ell f_y}{p_o}$$ ..... (Eq. 7–11(b))

7.6.2.2 *Contributions to A_t*

In calculating the term A_t/s in Equation Eq. 7–11(a), any closed stirrups provided for shear resistance or to satisfy minimum requirements may be included. A2

7.6.2.3 *Contributions to A_ℓ*

In calculating the term A_ℓ/p_o in Equation Eq. 7–11(a), longitudinal reinforcement used to resist flexure may be included provided that such reinforcement is anchored to provide full development. A2

7.6.3 *Torsional reinforcement details*

7.6.3.1 *Requirements*

Torsional reinforcement shall consist of closed stirrups perpendicular to the axis of the member combined with longitudinal bars.

7.6.3.2 *Maximum stirrup spacing*

Spacing of closed stirrups shall not exceed $p_o/8$, or 300 mm whichever is smaller.

7.6.3.3 *Maximum longitudinal bar spacing*

Spacing of longitudinal bars, distributed around the perimeter of the stirrups shall not exceed 300 mm centre-to-centre.

7.6.3.4 *Corner bar requirements*

At least one longitudinal bar or prestressed strand having a diameter equal to or greater than either s/16 or 10 mm shall be placed inside each corner of the closed stirrups. These corner bars or presstressing strands shall be anchored to provide full development A2

7.6.3.5 *Termination of torsional reinforcement*

Torsional reinforcement shall be provided at least a distance $p_o/2$ beyond the point of zero torsion.

7.6.3.6 *Anchoring of stirrups* A2

The closed stirrups shall be anchored by one of the appropriate methods detailed below:

(a) Welding, to give a continuous stirrup in accordance with 8.7.4.1(b);

(b) Anchored round a bar by a standard 135° hook, so that the tail is inside the concrete confined by the stirrup;

(c) Anchored by a corner bar by a standard 90° hook, where a flange protects the concrete on the outside of the hook from spalling.

7.6.3.7 *Torsional reinforcement in flanges*

Where flanged sections are used, in accordance with 7.6.1.7, closed stirrups and longitudinal bars shall be provided also in the overhanging parts of the flanges which have been considered in determining A_o and A_{co}.

7.6.4 *Design of reinforcement for torsion required for equilibrium* A2

7.6.4.1 *Design moment for torsion*

Where the torsional design action, T^*, exceeds $0.1\phi A_{co} t_c \sqrt{f_c'}$, (see 7.6.1.2), reinforcement shall be designed to resist a nominal torsional moment, T_n, equal to or greater than the larger of:

(a) $T_n = 0.44 A_{co} t_c \sqrt{f_c'} \left(1 + \dfrac{N^*}{0.33 A_g \sqrt{f_c'}} \right)$ or

(b) $T_n = T^*/\phi$

7.6.4.2 *Areas of closed stirrups and longitudinal reinforcement*

The minimum area, A_t, of a leg or legs of closed stirrups on one side of a member within a spacing of s along the member shall be equal to:

A2

$$A_t = \frac{T_n s}{2 A_o f_{yt}} \quad \text{(Eq. 7–12(a))}$$

The corresponding minimum area of longitudinal reinforcement, A_ℓ, around the perimeter p_o, shall be equal to:

$$A_\ell = \frac{T_n p_o}{2 A_o f_y} \quad \text{(Eq.7–12(b))}$$

The reinforcement areas, A_t and A_ℓ, shall be added to the areas of reinforcement required to resist flexure, shear and axial load, and the detailing of the reinforcement shall comply with 7.6.3. The reinforcement area, A_ℓ, shall be distributed symmetrically round the perimeter, p_o, except where modified by 7.6.4.3.

7.6.4.3 *Longitudinal torsional reinforcement reduction in compression zone*

In the flexural compression zone of a member the area of longitudinal torsional reinforcement required may be reduced by the amount equal to $\frac{M^*}{0.9df_y}$, where M^* is the design moment at the section acting in combination with T^*.

7.7 Shear-friction

7.7.1 *General*

The provisions of 7.7 shall apply when considering the shear transferred across a plane such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times or where there is a change in member cross section. The provisions of 7.7 do not need to be satisfied where the design does not contain construction joints and complies with the design requirements for shear in beams, columns or walls, or where a strut and tie analysis is used.

7.7.2 *Shear-friction design*

Design of cross sections subject to shear transfer as described in 7.7.1 shall be based on Equation 7–4 where V_n is calculated in accordance with the provisions of 7.7.4.

7.7.3 *Design approach*

A crack shall be assumed to occur along the shear plane considered. The required area of shear-friction reinforcement, A_{vf}, across the shear plane shall be designed using 7.7.4 or any other shear transfer design methods that result in the prediction of strength in substantial agreement with results of comprehensive tests.

7.7.4 *Shear-friction design method*

7.7.4.1 *Shear-friction reinforcement perpendicular to shear plane*

When shear-friction reinforcement is perpendicular to the shear plane, nominal shear strength, V_n, shall be computed by:

$V_n = (A_{vf} f_y + N^*)\mu$ (Eq. 7–13)

where N^* is the design force at the ultimate limit state acting normal to the shear plane and is positive for compression, and μ is the coefficient of friction in accordance with 7.7.4.3.

7.7.4.2 *Shear-friction reinforcement inclined to shear plane*

When shear-friction reinforcement is inclined to the shear plane, such that the shear force produces tension in shear-friction reinforcement, shear strength V_n shall be computed by:

$V_n = A_{vf} f_y(\mu \sin \alpha_f + \cos \alpha_f) + N^* \mu$ (Eq. 7–14)

where α_f is the angle between shear-friction reinforcement and the shear plane.

7.7.4.3 *Coefficient of friction*

The coefficient of friction μ in Equations 7–13 and 7–14 shall be:

(a) Concrete placed monolithically 1.4λ

(b) Concrete placed against hardened concrete with surface intentionally roughened as specified in 7.7.9 ..... 1.0λ

(c) Concrete placed against hardened concrete not intentionally roughened 0.6λ

(d) Concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars (see 7.7.10) 0.7λ

where

λ = 1.0 for normal density concrete,
= 0.85 for sand-lightweight concrete, and
= 0.75 for all-lightweight concrete.

Linear interpolation shall be permitted when partial sand replacement is used.

7.7.5 *Maximum shear strength*

Shear strength V_n shall not exceed the maximum permissible shear stress given in 7.5.2 times A_{cw}, where A_{cw} is the area of concrete section resisting the shear but neglecting any concrete cover on the tension side of the flexural tension reinforcement.

7.7.6 *Design yield strength of shear-friction reinforcement*

Design yield strength of shear-friction reinforcement shall not exceed 500 MPa.

7.7.7 *Reinforcement for net tension across shear plane*

Net tension across shear plane shall be resisted by additional reinforcement. Permanent net compression across shear plane may be taken as additive to the force in the shear-friction reinforcement $A_{vf}f_y$ when calculating required A_{vf}.

7.7.8 *Shear-friction reinforcement*

Shear-friction reinforcement shall be suitably distributed across the assumed crack and shall be adequately anchored to develop the yield strength on both sides by embedment, hooks, or welding to special devices. All reinforcement within the effective section, resisting flexure and axial load, normal to and crossing the potential sliding plane, may be included in determining A_{vf}.

7.7.9 *Concrete placed against previously hardened concrete*

For the purpose of 7.7, when concrete is placed against previously hardened concrete, the interface for shear transfer shall be clean and free of laitance. If μ is assumed equal to 1.0λ, the interface shall be roughened to a full amplitude of approximately 2 mm.

7.7.10 *Concrete placed against as-rolled structural steel*

When shear is transferred between as-rolled structural steel and concrete headed studs or welded reinforcing bars shall be used. The steel shall be clean and free of paint.

7.7.11 *Additional design requirements for members designed for earthquake effects*

Where shear resistance in members subjected to earthquake forces is to be provided by shear-friction and it is shown that response at the critical shear plane remains in the elastic range the provisions of 7.7 shall be used. The provisions of 7.7 shall not be used in potential plastic hinge regions of beams and columns. In these regions protection against sliding shear shall be in accordance with 9.4.4.1.4 and 9.4.4.1.5.

NOTES

8 STRESS DEVELOPMENT, DETAILING AND SPLICING OF REINFORCEMENT AND TENDONS

8.1 Notation

A_b area of an individual bar, mm^2

A_{sp} area of flexural reinforcement provided, mm^2

A_{sr} area of flexural reinforcement required, mm^2

A_{tr} smaller of area of transverse reinforcement within a spacing s crossing plane of splitting normal to concrete surface containing extreme tension fibres, or total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2. If longitudinal bars are enclosed within spiral or circular hoop reinforcement, $A_{tr} = A_t$ when $n \leq 6$.

A_v area of shear reinforcement within a distance s, mm^2

A_w area of an individual wire to be developed or spliced, mm^2

b_w web width, or diameter of circular section, mm

c_b neutral axis depth corresponding to balanced conditions, mm

c_m the smaller of the concrete cover or the clear distance between bars, mm

d distance from extreme compression fibre to centroid of tension reinforcement, mm

d_b nominal diameter of bar, wire or prestressing strand, or in a bundle, the diameter of a bar of equivalent area, mm

d_i diameter of bend measured to the inside of the bar, mm

f'_c specified compressive strength of concrete, MPa

f_{ps} calculated stress in prestressing steel at design load, MPa

f_s stress in reinforcing bar, MPa

f_{se} effective stress in prestressing steel after losses, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yt} lower characteristic yield strength of transverse reinforcement, MPa

L_b distance from critical section to start of bend, mm

L_d development length, mm

L_{db} basic development length of a straight bar, mm

L_{dh} development length of hooked bars, equal to straight embedment between critical section and point of tangency of hook, plus bend radius, plus one bar diameter, mm. (Refer to Figure 8.1)

L_{ds} splice length of bars in non-contact lap splices in flexural members, mm

M_n nominal flexural strength of section, N mm

n number of bars uniformly spaced around circular sections, or the number of longitudinal bars in the layer through which a potential plane of splitting would pass

s maximum spacing of transverse reinforcement within L_d, or spacing of stirrups or ties or spacing of successive turns of a spiral, all measured centre-to-centre, mm

s_b for a particular bar or group of bars in contact, the centre-to-centre distance or, measured perpendicular to the plane of the bend, to the adjacent bar or group of bars or, for a bar or group of bars adjacent to the face of the member, the cover plus one half of d_b, mm

s_L clear distance between bars of a non-contact lap splice, mm

s_w spacing of wires to be developed or spliced, mm

V^* design shear force at section at the ultimate limit state, N

α_1, α_2 parameters used in determining development lengths for standard hooks

α_a, α_b, α_c, α_d, α_e parameters used in determining development lengths for straight reinforcing bars

β_b ratio of area of reinforcement to be cut off to total area of tension reinforcement at the section, including those bars which are to be cut off

8.2 Scope

This section presents general provisions that shall apply to detailing of reinforcement and tendons, including spacing and design of anchorage, development and splices.

Provisions specific to particular elements are presented within the sections specific to those elements.

8.3 Spacing of reinforcement

8.3.1 *Clear distance between parallel bars*

The clear distance between parallel reinforcing bars in a layer shall be equal to or greater than the largest of the nominal diameter of the bars, or 25 mm, except that bars in slabs may be placed in two bar bundles.

8.3.2 *Nominal maximum size of aggregate*

The nominal maximum size of the aggregate shall be equal to or less than three-quarters of the minimum clear spacing between individual reinforcing bars or bundles or pre-tensioning tendons or post-tensioning ducts.

8.3.3 *Placement of parallel bars in layers*

Where parallel reinforcement is placed in two or more layers in beams, the bars in the upper layers shall be placed directly above those in the bottom layer with the clear distance between layers shall be the larger of the nominal diameter of the bars or 25 mm.

8.3.4 *Bundled bars*

Except in slabs, groups of parallel reinforcing bars bundled in contact and assumed to act as a unit shall only be used when the bundles are within the perimeter stirrups or ties. Bundles shall not contain any more than four bars. Bars larger than 32 mm shall not be bundled in beams or girders. Individual bars in a bundle cut off within the span of flexural members shall terminate at different points with at least 40 bar diameters stagger. Where spacing limitations and minimum clear cover are based on bar size, a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

8.3.5 *Spacing of principal reinforcement in walls and slabs*

The requirements for the spacing of reinforcement in walls is given by 11.3.10, 11.3.11.2, 11.4.6.3 and 11.4.6.5. The requirements for the spacing of reinforcement in slabs is given by 9.3.8.3, 9.3.9.4.12, 9.3.9.6.2, 12.5.6.3, 12.7.4 and 12.8.2.3.

8.3.6 *Spacing of outer bars in bridge decks or abutment walls*

In bridge decks or abutment walls, the maximum spacing between adjacent bars in the outermost layer shall be 300 mm.

8.3.7 *Spacing between longitudinal bars in compression members*

In spirally reinforced and tied compression members, the clear distance between longitudinal bars shall be equal to or greater than $1.5d_b$, or 40 mm.

8.3.8 *Spacing between splices*

The limit on clear distance between bars shall also apply to the clear distance between a contact lap splice and adjacent splices or bars.

8.3.9 *Spacing between pre-tensioning reinforcement*

Except for hollow-core floor systems as provided for in 8.3.9, the clear distances between pre-tensioning reinforcement at each end of the member shall be equal to or greater than 4 d_b for individual wires or $3d_b$ for strands. Closer vertical spacing and bundling of strands is permitted in the middle portion of the spans, but the requirements of 8.3.3 shall be satisfied. In hollow-core floor systems the clear distance between prestressing strands shall be equal to or greater than $2d_b$.

8.3.10 *Bundles of ducts for post-tensioned steel*

Ducts for post-tensioning steel may be bundled if it can be shown that the concrete can be satisfactorily placed and provision is made to prevent the steel, when tensioned, from breaking through the duct.

8.4 Bending of reinforcement

8.4.1 *Compliance with NZS 3109*

Bending and re-bending of reinforcing bars shall comply with the provisions of NZS 3109 including its amendments.

8.4.2 *Bending of steel bar reinforcement*

8.4.2.1 *Minimum bend diameter for main bars*

The diameter of bend, measured to the inside of the bar, shall be equal to or greater than the greater of the appropriate value given in Table 8.1 for steel reinforcement manufactured to AS/NZS 4671 or the value given by Equation 8–1, except that Equation 8–1 need not apply in the case where two transverse bars of d_b greater than or equal to the bar being bent are placed in contact with the inside of the bend or where the stress at the start of the bend is less than $f_y/2$. Where transverse bars are required they shall extend for a minimum distance of $3d_b$ beyond the plane of the last bent bar.

Table 8.1 – Minimum diameters of bend

f_y (MPa)	Bar diameter, d_b (mm)	Minimum diameter of bend, d_i (mm)
300 or 500	6 – 20	5 d_b
	24 – 40	6 d_b

The diameter of bend measured to the inside of the bar shall be equal to or greater than:

$$d_i \geq 0.92\left(0.5+\frac{d_b}{s_b}\right)\frac{f_s d_b}{f_c'} \quad \text{.....(Eq. 8–1)}$$

where f_s is the stress in the bar at the start of the bend. This may be taken as f_y, or a lower value if this justified by a rational analysis which allows for the influence of diagonal cracking on stress.

8.4.2.2 *Minimum bend diameter in fatigue situations*

In members subjected to frequently repetitive loading situations, the minimum diameter of bends in flexural reinforcement shall comply with 2.5.2.2. A2

8.4.2.3 *Stirrup and tie bends*

The inside diameter of bends of stirrups shall be greater than or equal to the diameter of the largest enclosed bar, and greater than or equal to the values given in Table 8.2.

Table 8.2 – Minimum diameters of bends for stirrups and ties

f_y (MPa)	Stirrup or tie diameter d_b (mm)	Minimum diameter of bend, d_i (mm)	
		Plain bars	Deformed bars
300 or 500	6 – 20	2 d_b	4 d_b
	24 – 40	3 d_b	6 d_b

8.4.2.4 *Bends in galvanised deformed bars*

Where deformed bars are galvanised before bending, the minimum bend diameter shall be:

(a) $5d_b$ for bar diameters of 16 mm or less;

(b) $8d_b$ for bar diameters of 20 mm or greater.

8.4.3 *Bending of welded wire fabric*

The inside diameter of bends in welded wire fabric, plain or deformed, shall be equal to or greater than four wire diameters for deformed wire larger than 7 mm and two wire diameters for all other wires. Bends with an inside diameter of less than eight wire diameters shall be equal to or greater than four wire diameters from the nearest welded intersection.

8.5 Welding of reinforcement

8.5.1 *Compliance with AS/NZS 1554:Part 3*

Except as provided herein, all welding shall conform to AS/NZS 1554:Part 3. In the design and execution of welding of reinforcing bar, appropriate account shall be taken of the process of manufacture.

8.5.2 *In-line quenched and tempered steel bars*

Welding, including tack welding, and hot bending of bars that have been manufactured by the in-line quenched and tempered process shall not be permitted.

8.5.3 *Welds in proximity to bends*

Welds in reinforcing bars shall be at least $3d_b$ away from the commencement of bends or that part of a bar which has been bent and re-straightened in accordance with NZS 3109.

8.6 Development of reinforcement

8.6.1 *Development of reinforcement – General*

Calculated tension or compression in reinforcement at each section of a reinforced concrete member shall be developed on each side of that section by embedment length or end anchorage or a combination thereof. Hooks may be used in developing bars in tension.

8.6.2 *Development of shear and torsion reinforcement*

The development of shear and torsion reinforcement shall comply with the relevant requirements of 7.5.7 and 7.6.3 respectively.

8.6.3 *Development length of deformed bars and deformed wire in tension*

8.6.3.1 *Development length in tension*

The development length, L_d, of deformed bars and wire in tension shall be calculated from either 8.6.3.2 or 8.6.3.3, but L_d shall be equal to or greater than 300 mm.

8.6.3.2 *Basic development length in tension*

Unless a more detailed determination of L_d is made in accordance with 8.6.3.3, the development length, L_{db} shall be calculated from:

$$L_{db} = \frac{(0.5\alpha_a f_y)}{\sqrt{f_c'}} d_b \quad \text{.....(Eq. 8–2)}$$

where α_a = 1.3 for top reinforcement where more than 300 mm of fresh concrete is cast in the member below the bar, or 1.0 for all other cases.

The value of f_c' used in Equation 8–2 shall not exceed 70 MPa.

8.6.3.3 *Refined development length in tension*

The development length, L_d, in tension may be determined from:

$$L_d = \frac{\alpha_b}{\alpha_c \alpha_d} L_{db} \geq 300\,\text{mm} \quad \text{(Eq. 8–3)}$$

with α_b, α_c and α_d being defined as follows:

(a) Reinforcement provided in a flexural member (not subjected to seismic forces nor required for temperature or shrinkage in restrained members) in excess of that required:

$$\alpha_b = A_{sr}/A_{sp} \quad \text{(Eq. 8–4)}$$

(b) When cover to bars in excess of $1.5d_b$ or clear distance between adjacent bars in excess of $1.5\,d_b$ is provided:

$$\alpha_c = 1 + 0.5\left(\frac{c_m}{d_b} - 1.5\right) \quad \text{(Eq. 8–5)}$$

with the limitation of $1.0 \leq \alpha_c \leq 1.5$

where c_m = the lesser of the concrete cover or the clear distance between bars.

(c) When transverse reinforcement with at least 3 bars, spaced less than $8d_b$, transverse to the bar being developed, and outside it, are provided within L_d:

$$\alpha_d = 1 + \sqrt{\left(\frac{A_{tr}}{s}\right)\left(\frac{f_{yt}}{80nd_b}\right)} \quad \text{(Eq. 8–6)}$$

with the limitation of $1.0 \leq \alpha_d \leq 1.5$

Transverse reinforcement used for shear, flexure or temperature may be included in A_{tr}.

8.6.4 ***Development length of plain bars and plain wire in tension***

The development of plain bars and wire in tension shall rely on hooks. The development length shall be twice the value for L_{dh} calculated from Equation 8–12.

8.6.5 ***Development length of deformed bars and deformed wire in compression***

8.6.5.1 *Development length in compression*

Development length L_d of deformed bars in compression shall be computed from either 8.6.5.2 or 8.6.5.3, but L_d must be greater than 200 mm.

8.6.5.2 *Basic development length in compression*

Unless a more detailed determination of L_d is made in accordance with 8.6.5.3 the development length in compression, L_{db}, shall be calculated from:

$$L_{db} = \frac{0.22 f_y}{\sqrt{f'_c}} d_b \quad \text{(Eq. 8–7)}$$

with limitations of

$$L_{db} \geq 0.040 f_y d_b \geq 200\,\text{mm} \quad \text{(Eq. 8–8)}$$

The value of f_c' used in Equation 8–7 shall not exceed 70 MPa.

8.6.5.3 *Refined development length in compression*

The development length in compression, L_d, may be determined from:

$$L_d = \alpha_b \alpha_e L_{db} \quad \text{(Eq. 8–9)}$$

with α_b as defined in 8.6.3.3(a) and α_e as follows:

When transverse reinforcement with at least three bars, transverse to the bar being developed and outside it, are provided within L_{db}, and $\frac{A_{tr}}{s} \geq \frac{A_b}{600}$

α_e = 0.75, or
= 1.0 for all other cases.

8.6.6 *Development length of plain bars and plain wires in compression*

The development length for plain bars and wires in compression shall be twice the calculated value L_d or L_{db} for a deformed bar or wire.

8.6.7 *Development of bundled bars*

Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased by 20 % for a three-bar bundle, and 33 % for a four-bar bundle.

8.6.8 *Development of welded plain and deformed wire fabric in tension*

8.6.8.1 *Development length of wire fabric*

Development length, L_d, of welded plain and deformed wire fabric measured from the point of critical section to the end of the wire shall be computed from either 8.6.8.2 or 8.6.8.3.

8.6.8.2 *Development length of welded wire fabric – cross wires considered*

The yield strength of plain and deformed wires of welded wire fabric shall be considered developed by embedding at least two cross wires, with the first one equal to or greater than 50 mm from the critical section. However, development length L_d measured from the critical section to the outermost cross wire shall be equal to or greater than 100 mm:

$$L_d \geq \frac{3.25 \alpha_b A_w f_y}{s_w \sqrt{f_c'}} \quad \text{(Eq. 8–10)}$$

where α_b is given by 8.6.3.3(a), but L_d shall be equal to or greater than 150 mm for plain wire fabric or greater than 100 mm for deformed wire fabric.

8.6.8.3 *Development length of welded wire fabric – cross wires not considered*

The development length of welded deformed and plain wire fabric, with no cross wires or when the cross wires within the development length as required by 8.6.8.2 are ignored, shall be determined by 8.6.3 or 8.6.4 as appropriate and shall be equal to or greater than 200 mm.

8.6.9 *Development of prestressing strand*

8.6.9.1 *Development length of pre-tensioning strand*

Three or seven-wire pre-tensioning strand shall be bonded beyond the critical section for a development length given by:

$$L_d \geq \left(f_{ps} - \frac{2}{3} f_{se} \right) \frac{d_b}{7} \quad \text{(Eq. 8–11)}$$

8.6.9.2 *Development of pre-stressing strand*

Where bonding of a strand does not extend to the end of a member, the bonded development length specified in 8.6.9.1 shall be doubled.

8.6.10 *Standard hooks*

8.6.10.1 *Standard hooks – definition*

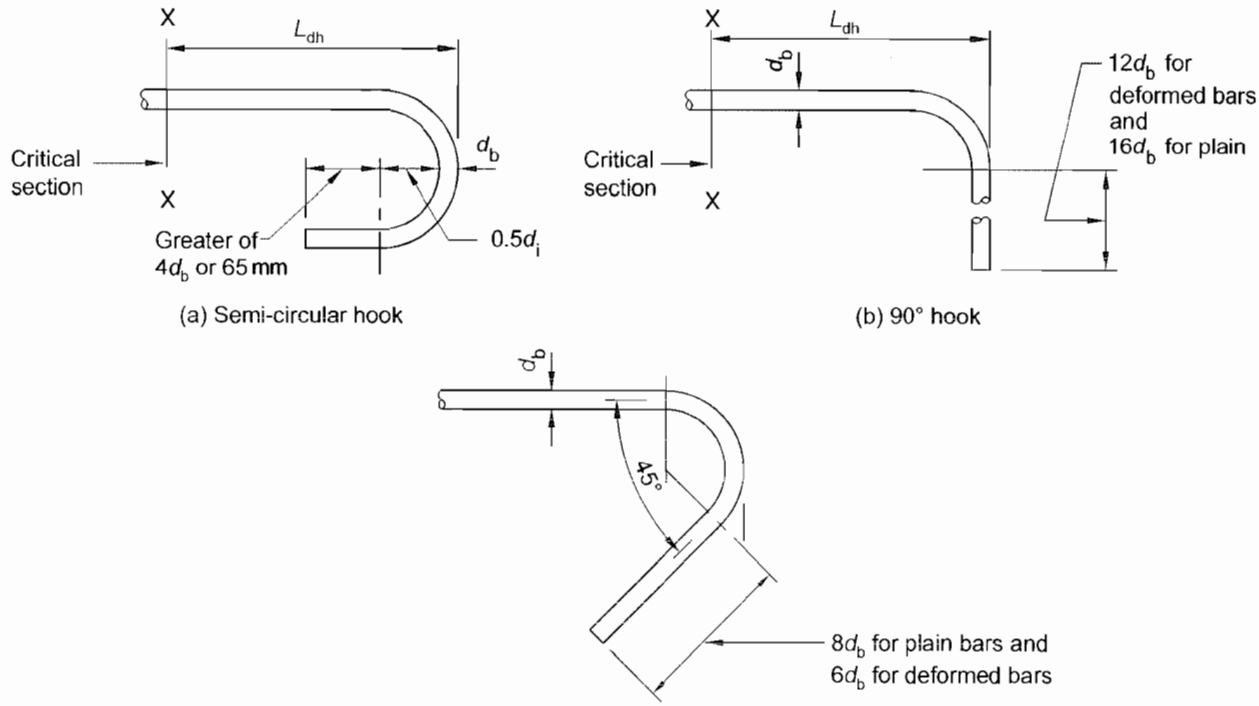
The term "standard hook" as used herein shall mean either:

(a) A semi-circular turn plus an extension of at least four bar diameters but equal to or greater than 65 mm at the free end of the bar; or

(b) A 90° turn plus an extension of at least 12 bar diameters at the free end of the bar for a deformed bar and 16 bar diameters for a plain bar; or A2

(c) A stirrup hook, which is defined as a 135° turn around a longitudinal bar plus an extension of at least eight stirrup bar diameters for plain bars and six stirrup bar diameters for deformed bars at the free end of the bar embedded in the core concrete of the member.

The standard hooks defined in this clause are illustrated in Figure 8.1.



A2

Figure 8.1 – Standard hooks

8.6.10.2 *Bars > 32 mm in diameter*

Bars with diameter greater than 32 mm shall not be developed in tension by the use of standard hooks.

8.6.10.3 *Development length of standard hooks in tension*

8.6.10.3.1 *Calculation of development length for hooked bars*

For the following two situations described in (a) and (b), the development length, L_{dh}, for hooks in tension shall be determined from Equation 8–12:

(a) Where the bar is anchored by a standard hook inside a volume of concrete confined by closed ties, spirals or stirrups perpendicular to the plane of the hook;

(b) Where the bar, is anchored by a standard hook inside a volume of concrete that is not confined by reinforcement perpendicular to the plane of the hook, but

(i) The spacing between that bar and any adjacent bar or fixing loaded in a similar direction is greater than or equal to three times d_b over L_{dh} for that bar; and

(ii) The distance normal to the axis of the bar to the side or edge of the element is greater than or equal to two times d_b over L_{dh} for that bar.

$$L_{dh} = 0.24\alpha_b\alpha_1\alpha_2\frac{f_y d_b}{\sqrt{f_c'}} \geq 8d_b \quad \text{.....(Eq. 8–12)}$$

where

f_c' shall not be taken greater than 70 MPa

α_b is given by 8.6.3.3 (a)

α_1 = 0.7 for 32 mm bars or smaller with side cover normal to the plane of the hook ≥ 60 mm, and cover on the tail extension of 90° hooks equal to or greater than 40 mm

= 1.0 for all other cases

α_2 = 0.8 where confined by closed stirrups or hoops spaced at $6d_b$ or less and which satisfy the relationship $\frac{A_{tr}}{s} \geq \frac{A_b}{1000}$

= 1.0 for all other cases

8.6.10.3.2 *Determination of development length where not covered by 8.6.10.3.1*

For situations other than as described by 8.6.10.3.1(a) and (b), the development length of a hook shall be determined from a rational analysis or suitable testing that takes into account the effects of the proximity of the anchored bar to edges of elements and to other loaded embedded items.

8.6.10.3.3 *Development length of standard hooks anchoring around transverse bars*

The development length L_{dh} of a deformed bar terminating in a standard hook as determined from 8.6.10.3 may be reduced by 20 %, provided that two transverse bars having a diameter equal to or larger than that of the bent bar are placed in contact with the inside of the bend and extend for a distance equal to or greater than $3d_b$ beyond the centreline of the bent bar.

8.6.10.4 *Hooks in compression*

Hooks shall not be considered effective in developing reinforcement in compression.

8.6.11 *Mechanical anchorage*

8.6.11.1 *General*

Any mechanical device used alone as an anchorage, or used in combination with an embedment length beyond the point of maximum stress in the bar, shall be capable of developing the upper characteristic breaking strength of the reinforcing bar without damage to the concrete or overall deformation of the anchorage.

8.6.11.2 *Upper bound breaking strength for the reinforcing bar – definition*

The upper characteristic breaking strength of the reinforcing bar may be derived from 1.15 times the upper characteristic yield strength specified by AS/NZS 4671, or otherwise shall be determined from an appropriate testing programme.

8.6.11.3 *Adequacy of mechanical devices*

Mechanical anchorage systems relying on interconnecting threads or mechanical interlock with the bar deformations for attachment of the anchorage to the bar shall meet both the permanent extension and fatigue strength criteria of 8.7.5.2.

8.6.12 *Development of flexural reinforcement*

8.6.12.1 *Bending across the web*

Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

8.6.12.2 *Critical sections*

Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent.

8.6.12.3 *Extension of tension reinforcement*

Except at supports of simply supported spans and at the free end of cantilevers, tension reinforcement shall extend beyond the point at which, according to the bending moment envelope and standard flexural theory, it is:

(a) Required at maximum stress for a distance equal to the development length, L_d, plus the effective depth of the member, and

(b) No longer required to resist flexure for a distance of 1.3 times the effective depth of the member.

8.6.12.4 *Termination in a tension zone*

Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

(a) Shear at the cut-off point is less than two-thirds of the shear strength provided by the concrete; or

(b) The shear strength provided by the web reinforcement , V_s, measured for a distance of 1.3d along the terminating bar from the cutoff point is equal to or greater than:

$$V_s = 1.2\frac{\sqrt{f_c'}}{16} b_w d \quad \text{.....(Eq. 8–13)}$$

and the spacing, s, of stirrups or ties is equal to or less than the smaller of $d/2$ or $\frac{d}{8\beta_b}$.

8.6.12.5 *End anchorage in flexural members*

Adequate end anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets, deep flexural members; or members in which tension reinforcement is not parallel to the compression face.

8.6.13 *Development of positive moment reinforcement in tension*

8.6.13.1 *Limitation in area of bars*

At least one-third the maximum positive moment reinforcement in simply supported members and one quarter the maximum positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm unless a lesser distance is demonstrated by test to be adequate and to provide the structural robustness required by AS/NZS 1170.0.

8.6.13.2 *Critical sections*

Where a flexural member is part of a primary horizontal force-resisting system, positive moment reinforcement required to be extended into the support by 8.6.13.1 shall be anchored to develop the lower characteristic yield strength, f_y, in tension at the face of support.

8.6.13.3 *Limitation in diameter of bars at simple supports*

The positive tension reinforcement at simple supports shall be limited in diameter to enable the bars extending to the free end of the member to be fully developed from a point M_n/V^* from the centre of the

support. The value of M_n / V^* shall be calculated at the centre of the support and may be increased by 30 % when the ends of reinforcement at the support are confined by a compressive reaction.

8.6.13.4 *Limitation in diameter of bars at points of inflection*

The positive (and negative) tension reinforcement at points of inflection shall be limited in diameter to enable the bars, from a point M_n / V^* from the point of inflection, to be fully developed satisfying the requirements that:

$$L_d \leq \frac{M_n}{V^*} + 12 d_b \quad \text{.....(Eq. 8–14)}$$

and

$$L_d \leq \frac{M_n}{V^*} + d \quad \text{.....(Eq. 8–15)}$$

8.6.14 *Development of negative moment reinforcement in tension*

8.6.14.1 *Anchorage of bars*

Negative moment reinforcement in a continuous, restrained or cantilever member, or in any member of a rigid jointed frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

8.6.14.2 *Embedment length adjacent to supports*

Negative moment reinforcement shall have an embedment length into the span as required by 8.6.1 and 8.6.12.3.

8.6.14.3 *Embedment length beyond the point of inflection*

At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection, according to the appropriate bending moment envelope, for a distance equal to or greater than 1.3 times the effective depth of the member.

8.6.14.4 *Limitation in diameter of bars*

The requirements of 8.6.13.4 at points of inflection for negative reinforcement shall be satisfied.

8.7 Splices in reinforcement

8.7.1 *General*

Splices in reinforcement shall be shown on the design drawings or specified in the specifications.

8.7.2 *Lap splices of bars and wire in tension*

8.7.2.1 *Bar sizes of lap splices*

Lap splices shall not be used for bars larger than 40 mm in diameter.

8.7.2.2 *Lap splices of bundled bars*

Lap splices of bundled bars shall be based on the lap splice length required for individual bars of the same size as the bars spliced, and such individual splices within the bundle shall not overlap each other. The length of lap, as prescribed in 8.7.2.3 or 8.7.3, shall be increased by 20 % for a three-bar bundle and 33 % for a four-bar bundle.

8.7.2.3 *Length of lap splices of deformed bars or wire*

The minimum length for lap splices of deformed bars and deformed wire in tension shall be equal to or greater than the development length, L_d, in 8.6.3. Plain straight bars or wires shall not be spliced by lapping unless using hooks or other anchorages.

8.7.2.4 *Length of lap splices of hooked plain bars or wire*

The length of lap splices for hooked plain bars or wire, including those permitted to be used by 5.3.1, with a standard hook shall be equal to or greater than the development length required by 8.6.4. For bars with 50 mm of cover concrete or less, hooks shall be in a plane at a right angle to the adjacent concrete surface. Such splices shall not be used in potential plastic hinge regions of members.

8.7.2.5 *Length of non-contact lap splices*

Bars spliced by non-contact lap splices in flexural members spaced transversely farther apart than $3d_b$ shall have splice length, L_{ds}, given by $L_{ds} \geq L_d + 1.5\, s_L$.

8.7.2.6 *Strength developed at sections*

In computing the strength developed at each section, spliced bars shall be rated at the specified splice strength.

8.7.2.7 *Strength of bars where cut off*

Bars cut off near the section under consideration taking the requirements of 8.6.12.3 into account shall be rated only at a fraction of f_y, defined by the ratio of the embedded length past this section to the required development length.

8.7.2.8 *Lap splices of stirrups, ties and hoops*

Stirrups, ties and rectangular hoops in beams, columns, piers, beam column joints or walls may be spliced by lapping provided that the requirements in either (a) or (b) are satisfied: A2

(a) Lapping bars shall be terminated with standard hooks in accordance with 8.4.2.1 and 8.6.10.1, and the splice length shall be:
   (i) For plain bars, equal to or greater than the development length required by 8.6.4;
   (ii) For deformed bars, equal to or greater than L_{dh} in 8.6.10.3.

   When the lapped splice is located in cover concrete, the hooks shall be placed in a plane at right angles to the surface of the concrete. When located in a plastic region, the hooks shall be anchored around longitudinal reinforcement of at least equal or greater diameter.

(b) Straight lapped splices with deformed bars may be used where not specifically excluded in (i) to (iii) below:
   (i) Straight lapped splices shall not be used in potential ductile or limited ductile plastic regions, or in beam column joints located next to potential ductile or limited ductile plastic regions;
   (ii) Straight lapped splices shall not be used in cover concrete where the reinforcement is required to provide confinement to the concrete;
   (iii) Straight lapped splices shall not be used in cover concrete where the shear stress due to shear and torsion exceeds $0.5\sqrt{f'_c}$.

8.7.3 ***Lap splices of bars and wires in compression***

8.7.3.1 *General*

The minimum length of a lap splice in compression shall be the development length in compression L_d, in accordance with 8.6.5 and 8.6.6, but equal to or greater than $0.069 f_y d_b$ for f_y of 430 MPa or less, or $(0.12\, f_y - 22)\, d_b$ for f_y greater than 430 MPa, or 300 mm.

8.7.3.2 *Lap splices in compression with stirrups and ties*

In compression members with stirrups and ties where at least three sets of ties are present over the length of the lap, and

$$\frac{A_{tr}}{s} \geq \frac{A_b}{1000} \qquad \text{(Eq. 8–16)}$$

or where transverse reinforcement as required by either 10.4.7.4.3 or 10.4.7.4.5 has been provided, a lap length of 0.8 times that specified in 8.7.3.1 may be used but the lap length shall be equal to or greater than 300 mm.

8.7.3.3 *Lap splices in compression with spiral reinforcement*

In spirally reinforced compression members, if at least three turns of spiral are present over the length of the lap, and

$$\frac{A_{tr}}{s} \geq \frac{nA_b}{6000} \quad \text{(Eq. 8–17)}$$

a lap length of 0.8 times that specified in 8.7.3.1 may be used, but the lap length shall be equal to or greater than 300 mm.

8.7.4 *Welded splices*

8.7.4.1 *Classification of welded splices*

Welded splices shall be classified as follows:

(a) A "full strength" welded splice is one in which the bars are butt welded to develop in tension the breaking strength of the bar;

(b) A "high strength" welded splice is one in which the bars are lap welded or butt welded to develop the lower characteristic yield strength of the bar or better.

8.7.4.2 *Limitations on the classification of welded splices for grade > 450 MPa reinforcement*

Butt welded splices in reinforcement with a lower characteristic yield stress of more than 450 MPa shall not be classified as "full strength" unless either:

(a) Yielding of the reinforcement will not occur; or

(b) Proof testing using a portion of the actual bar to be welded and the selected welding procedure, demonstrates that failure of the bar occurs away from the weld.

8.7.4.3 *Exceptions for welded splices*

The requirements 8.7.4.1(b) may be waived when the conditions of 8.7.5.4 are satisfied.

8.7.5 *Mechanical connections*

8.7.5.1 *Definition of mechanical connection*

A mechanical connection is defined as a connection that relies on interlocking threads or mechanical interlock with the bar deformations to develop the connection capacity.

8.7.5.2 *Performance requirements for mechanical connections*

Mechanical connections shall:

(a) develop in tension or compression, as required, not less than the upper characteristic breaking strength as defined in 8.6.11.2 of the bar;

(b) when tested in tension or compression, as appropriate, to the application, exhibit a change in length at a stress of $0.7f_y$ in the bar, measured over the length of the coupler, of less than twice that of an equal length of unspliced bar;

(c) satisfy the requirements of 2.5.2.2 when used in situations where fatigue may develop.

8.7.5.3 *Use of welded splices and mechanical connections*

Welded splices in tension or compression shall meet the requirements of 8.7.4.1 (a) or (b).

Mechanical connections in tension or compression shall meet the requirements of 8.7.5.2.

8.7.5.4 *Use of welded splices and mechanical connections – an exception*

The requirements of 8.7.4.1(b) and 8.7.5.2, as appropriate, may be waived when splices:

(a) Are staggered at least 600 mm; and

(b) Can develop at least twice the calculated force in the bars to be spliced at the section; and

(c) Can develop equal to or greater than 0.7 f_y based on the total area of effective bars across the section; and

(d) Where the level of any resulting premature cracking is not likely to affect the performance of the structure, then the change of length shall be not more than six times that of an equal length of unspliced bar.

8.7.5.5 *Identification and marking*

Each coupler or coupling sleeve shall be legibly and durably marked with the identification of the manufacturer and the nominal bar size for which it is intended. Each coupler or coupling sleeve shall be traceable back to its production data and production batch.

8.7.5.6 *Installation*

The method of installation of mechanical connection systems shall be specified for all conditions that arise on a job site. This may be by reference to manufacturers' written instructions. Connection systems that rely on a minimum length of engagement between the coupler or coupling sleeve and the bar for the development of the connection strength shall incorporate a system for positively locating the coupler or coupling sleeve and defining when adequate engagement has been achieved.

8.7.6 *Splices of welded plain or deformed wire fabric*

Lap splices shall be detailed by satisfying one of the following conditions:

(a) The overlap measurement between outermost cross wires of each fabric sheet is equal to or greater than the spacing of cross wires plus 50 mm, nor less than 1.5 L_d or 150 mm whichever is greater, where L_d is the development length for f_y as given in 8.6.8.2; or

(b) When cross wires are ignored or no cross wires are present within the lapped length and the lap is a contact or near contact lap splice, the splice length shall be equal to or greater than L_d, where L_d is the development length given by 8.6.8.3.

8.8 Shrinkage and temperature reinforcement

8.8.1 *Floor and roof slab reinforcement*

Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in structural floor and roof slabs where the principal reinforcement extends in one direction only. At all sections where it is required, such reinforcement shall be developed for its lower characteristic yield strength in conformance with 8.6.1 or 8.7.2. Such reinforcement shall provide at least the ratio of reinforcement area to gross concrete area of $0.7/f_y$, but equal to or greater than 0.0014.

8.8.2 *Large members*

In a large member whose size is not governed by stress considerations, or where exact analysis is impractical, minimum reinforcement on all surfaces should be the greater of 1000 mm^2 per metre width in each direction, with bars not further apart than 300 mm, or, where appropriate, as required by 2.4.4.8.

8.9 Additional design requirements for structures designed for earthquake effects

8.9.1 *Splices in reinforcement*

8.9.1.1 *Placement of splices*

Full strength welded splices meeting the requirements of 8.7.4.1(a) may be used in any location. For all other splices the following restrictions apply:

(a) No portion of any splice shall be located within the beam/column joint region, or within one effective depth of the member from the critical section of a potential plastic hinge in a beam where stress reversals in spliced bars could occur;

(b) In a column framing top and bottom into beams or other moment resisting elements, the centre of the splice must be within the middle quarter of the storey height of the column unless it can be shown that a high level of protection is provided against the formation of plastic regions, as defined in Appendix D.

8.9.1.2 *Lap splices in region of reversing stresses*

Reinforcement in beams and columns shall not be spliced by lapping in a region where reversing stresses at the ultimate limit state may exceed $0.6f_y$ in tension or compression unless each spliced bar is confined by stirrup-ties so that:

$$\frac{A_{tr}}{s} \geq \frac{d_b f_y}{48 f_{yt}} \quad \text{.....(Eq.8–18)}$$

except that where there is no alternative load path in a structure for the forces being carried by an element in the event of failure of the element, lap splicing shall not be permitted at all.

8.9.1.3 *Requirements for welded splices or mechanical connections*

For welded splices or mechanical connections to be used in members that are subjected to seismic forces, such splices shall comply with 8.7.4.1 or 8.7.5.2. In addition to the requirements of 8.7.5.2, mechanical splices shall be tested through 8 full cycles of loading to a maximum stress of 0.95 f_y in the bar, and at maximum load in both tension and compression shall show a change of length, measured over the full length of the connection system, not more than 10 % in excess of the extension in an equal length of unspliced bar. Splices not satisfying this stiffness requirement shall be used only if they are staggered so that no more than two thirds of the reinforcement area is spliced within any 900 mm length of the member.

8.9.2 ***Development length***

For calculation of development length, the reduction provisions of 8.6.3.3(a), 8.6.8.2 and 8.6.10.3.1 by α_b (equal to A_{st}/A_{sp}) shall not apply.

9 DESIGN OF REINFORCED CONCRETE BEAMS AND ONE-WAY SLABS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

9.1 Notation

A_b area of longitudinal bar, mm^2

A_{cv} effective shear area, area used to calculate shear stress, mm^2 | A2

A_g gross area of column cross section, mm^2

A_s area of flexural tension reinforcement, mm^2

A_s' area of compression reinforcement, mm^2

A_{te} area of one leg of stirrup-tie, mm^2

A_v area of shear reinforcement perpendicular to the span within a distance s, mm^2

A_{vd} area of diagonal shear reinforcement, mm^2

A_{vh} area of shear reinforcement parallel to span, mm^2

b width of compression face of a member, mm

b_w width of web, mm

c_b distance from extreme compression fibre to neutral axis at balanced strain conditions, as defined in 7.4.2.8, mm

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement. (For circular sections, d need not be taken less than the distance from extreme compression fibre to centroid of tension reinforcement in opposite half of member), mm

d_b nominal diameter of longitudinal reinforcing bar, mm

f_c' specified compressive strength of concrete, MPa

f_{ct} average splitting tensile strength of lightweight aggregate concrete, MPa

f_s' compression stress in the bar on one side of joint zone, MPa | A2

f_y lower characteristic yield strength of longitudinal reinforcement, MPa

f_{yt} lower characteristic yield strength of transverse reinforcement, MPa

| A2

h overall depth, mm

h_{b1}, h_{b2} beam depths used for determining effective flange widths, mm

h_c overall depth of column, mm

h_g overall depth of girder, mm | A2

k_a factor allowing for the influence of aggregate size on shear strength

k_d factor allowing for the influence of member depth on shear strength

K_{cp} factor for additional long-term deflection

ℓ_y potential plastic region ductile detailing length, mm

L_n clear span of member measured from face of supports, mm

L_{nd} length of longitudinal projection of diagonal reinforcement in a diagonally reinforced coupling beam, but not exceeding the clear span of the beam, mm | A2

M^* design bending moment at section at ultimate limit state, N mm

N_o^* min design overstrength axial load determined by capacity design in accordance with appendix D, N

n number of directions of diagonal bars (one or two)

p ratio of tension reinforcement = A_s/bd

p' ratio of compression reinforcement = A_s'/bd

p_{max}, p_{min} maximum and minimum permitted values of the ratio of tension reinforcement computed using width of web

p_w $A_s/b_w d$

r factor defined in 9.4.4.1.4

s spacing of transverse reinforcement in direction parallel to longitudinal reinforcement, mm

s_2 spacing of shear or torsional reinforcement in perpendicular direction to longitudinal reinforcement

v_c shear resisted by concrete, MPa

V_c nominal shear strength provided by the concrete, N
V_{di} design shear force to be resisted by diagonal shear reinforcement at the ultimate limit state, N
V_n total nominal shear strength of cross section of beam, N
V_s nominal shear strength provided by the shear reinforcement, N
V^* design shear force at section at the ultimate limit state, N
V_o^* maximum shear force sustained when overstrength actions act in a member or adjacent member, N
α angle between inclined stirrups or bent-up bars and longitudinal axis of members
α_d factor in Equation 9–21
α_f factor in Equations 9–21 and 9–22
α_o factor in Equations 9–21 and 9–22
α_p factor in Equation 9–23
α_s factor in Equation 9–24
α_t factor in Equation 9–22
γ factor given by Equation 9–20
ΣA_b sum of areas of longitudinal bars, mm^2
δ_c calculated inter-storey deflection, mm
δ_m maximum permissible inter-storey deflection, mm
θ angle of compression diagonals
ϕ strength reduction factor (see 2.3.2.2)
$\phi_{o,fy}$ overstrength factor depending on reinforcement grade, see 2.6.5.5.

9.2 Scope

The provisions of this section shall apply to the design of reinforced concrete members for flexure and shear without axial force. The provisions for this and earlier sections are summarised in Table C9.3. The written requirements take precedence over Table C9.3. Beams containing plastic regions with sectional curvature ductility demands less than or equal to the limits for the nominally ductile plastic region defined in 2.6.1.3 shall meet the requirements of 9.3. Beams containing plastic regions designed for greater sectional curvature ductility than this shall meet the requirements of 9.3 as modified by 9.4. A2

9.3 General principles and design requirements for beams and one-way slabs

9.3.1 *General*

9.3.1.1 *Moments at supports for beams integral with supports*

For beams built integrally with supports, moments at faces of support may be used for the design of reinforcement.

9.3.1.2 *Effective width resisting compression of T-beams*

In T-beam construction, the slab and web shall be built integrally or otherwise effectively bonded together and the following requirements shall also be satisfied:

(a) The width of slab assumed to be effective as a T-beam flange resisting compressive stresses due to flexure shall be equal to or less than the width of the web plus one-quarter the span length of the beam, and the effective compressive overhanging slab width on each side of the web shall not exceed the smaller of:
   (i) Eight times the minimum slab thickness
   (ii) The total depth of the beam
   (iii) The clear distance between adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$.

   Where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam.

(b) For beams with a flange on one side only, the effective width of overhanging slab considered to be effective in resisting compressive stresses due to flexure shall be equal to or less than the smaller of:
   (i) One-eighth of the span length of the beam

(ii) Eight times the slab thickness

(iii) The depth of the beam

(iv) The clear distance between adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$.

Where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam.

9.3.1.3 *Effective moment of inertia in T- beam*

In calculating the effective moment of inertia of cracked sections, the effective width of the overhanging parts of flanged members shall be one-half of that given by either 9.3.1.2(a)or (b).

9.3.1.4 *Contribution of slab reinforcement to design strength of T and L beams*

In T- and L- beams built integrally with slabs, slab reinforcement, which is identified in either (a) or (b) as being in the effective overhanging flange may be considered to contribute to the flexural strength of the beam. A2

(a) The contribution of reinforcement in the slab is established on the basis of engineering principles, in which allowance is made for shear lag. Slab reinforcement, which is assumed to contribute to flexural strength, shall be effectively tied into the web of the beam by transverse reinforcement. A strut and tie analysis shall be made to demonstrate that this reinforcement is effectively anchored and that shear arising from the forces in this reinforcement can be transferred from the overhanging flange to the web of the beam.

(b) The contribution of the reinforcement in each overhanging flange to the flexural strength shall satisfy the following requirements:

(i) The tensile strength of the reinforcement in the effective overhanging flange shall not exceed 15 percent of the total flexural tensile strength of the beam at the section being considered;

(ii) Only reinforcement in the effective overhanging flange, which is parallel to the beam, shall be considered to contribute to the design flexural strength of the beam. The effective overhanging flange shall be taken as the distances measured from the face of the web to the smaller of:

(A) The overall depth of the beam;

(B) 8 times the minimum thickness of the slab;

(C) The clear distance between adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$ where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam;

(D) One eighth of the span of the beam;

(E) Where the beam is at right angles to the free edge of the slab a distance equal to half the distance between the free edge of the slab and the section in the beam that is being considered. This distance is measured from the face of the column, or web of beam if there is no column. If there is an edge beam this width may be increased by the web width of the edge beam.

9.3.1.5 *Floor finishes*

When a separate floor finish is placed on a slab it shall be assumed that the floor finish is not included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with the requirements of Sections 13 and 18.

9.3.1.6 *Deep beams*

9.3.1.6.1 *Definition*

Deep beams are members loaded on one face and supported on the opposite face, so that compression struts can develop between the loads and supports, and have either:

(a) Clear spans, L_n, equal to or less than 3.6 times the effective depth for simply supported or continuous beams; or A2

(b) Clear spans equal to or less than 1.8 times the effective depth for cantilevered beams.

9.3.1.6.2 *Design requirements*

Deep beams shall be designed taking into account non-linear distribution of strains or by using strut-and-tie models. Possible lateral buckling shall be considered. Design of deep beams shall be in accordance with 9.3.10.

9.3.2 *Strength of beams and one-way slabs in bending*

The design of beams and one-way slabs for flexure at the ultimate limit state shall be based on the assumptions given in 7.4 and on the satisfaction of applicable conditions of equilibrium and compatibility of strains.

9.3.3 *Strength of beams and in shear*

The design of beams and for shear at the ultimate limit state shall be in accordance with 7.5 and 9.3.9.

9.3.4 *Strength of beams in torsion*

The design of beams for torsion, shear, and flexure at the ultimate limit state shall be in accordance with 7.6.

9.3.5 *Distance between lateral supports of beams*

9.3.5.1 *Limits on lateral support spacing*

Spacing of lateral supports for a beam shall not exceed 50 times the least width, b, of the compression flange or face.

9.3.5.2 *Effects of load eccentricity on lateral support spacing*

Effects of lateral eccentricity of load shall be taken into account in determining the spacing of lateral supports.

9.3.6 *Control of flexural cracking*

9.3.6.1 *General*

Members subjected to flexure shall be designed to control cracking in accordance with 2.4.4.

9.3.6.2 *Beams and one-way slabs*

In beams and one-way slabs, the flexural tension reinforcement shall be well distributed across the zone of maximum tension in the member cross section and shall satisfy 2.4.4.

9.3.6.3 *Skin reinforcement*

If the depth of a member exceeds 1.0 m, longitudinal skin reinforcement shall be placed along the side faces in accordance with 2.4.4.5.

9.3.7 *Control of deflections*

9.3.7.1 *Minimum thickness*

The minimum thickness specified in 2.4.3 shall apply unless the calculation of deflection according to 6.8 indicates that lesser thickness may be used without adverse effects.

9.3.8 *Longitudinal reinforcement in beams and one-way slabs*

9.3.8.1 *Maximum longitudinal reinforcement in beams and one-way slabs*

For beams and slabs the amount and distribution of longitudinal reinforcement provided shall be such that at every section, the distance from the extreme compression fibre to the neutral axis is less than $0.75c_b$. Where moment redistribution in accordance with 6.3.7 at a section is utilised, the neutral axis depth shall also comply with 6.3.7.2(f).

9.3.8.2 *Minimum longitudinal reinforcement in beams and one-way slabs*

A2 **9.3.8.2.1** *Minimum reinforcement in beams*

At every section of a beam, except as provided in one of 9.3.8.2.2, 9.3.8.2.3 or 9.3.8.2.4, (where tension reinforcement is required by analysis), the reinforcement area A_s provided shall be greater than that given by:

$$A_s = \frac{\sqrt{f_c'}}{4f_y} b_w d \quad \text{.....(Eq. 9–1)}$$

but equal to or greater than 1.4 $b_w d/f_y$.

For beams where the flexural strength of the member contributes to the lateral force resistance of the structure, the minimum negative moment reinforcement consisting of two or more bars shall extend right through the span. The area of this reinforcement shall be equal to or greater than the larger of one-quarter of the maximum negative moment reinforcement in the beam, or the value given by Equation 9–1. A2

9.3.8.2.2 *Minimum reinforcement in statically determinate T-beams*

For a statically determinate T-beam with the flange in tension, where A_s includes the area of longitudinal reinforcement in flanges in accordance with 9.3.1.4 the reinforcement area A_s shall be greater than the value given by Equation 9–1 with b_w replaced by either $2b_w$ or the width of the flange, whichever is smaller.

9.3.8.2.3 *Reduced minimum reinforcement* A2

For beams where the flexural strength does not contribute to the lateral strength of the structure, the area of longitudinal reinforcement given by Equation 9–1 with its minimum limit of 1.4 $b_w d/f_y$, and by 9.3.8.2.2, may be reduced, provided that at every section of a beam, for positive and negative moment the area of reinforcement shall be at least one-third greater than that required by analysis.

9.3.8.2.4 *Minimum reinforcement in slabs and footings*

For structural slabs and footings of uniform thickness, the minimum area of principal reinforcement shall satisfy 9.3.8.2.1 and for reinforcement normal to the principal reinforcing and spacing of reinforcement shall be as required for shrinkage and temperature according to 8.8.

9.3.8.3 *Spacing of reinforcement in slabs*

The spacing of principal reinforcement in slabs shall not exceed the smaller of two times the slab thickness or 300 mm. For reinforcement perpendicular to the principal reinforcement, the maximum spacing of reinforcement shall not exceed the lesser of three times the slab thickness, 300 mm for bridges or 450 mm for buildings. The spacing of reinforcement in *in situ* concrete topping in floors containing precast units shall be equal to or less than 400 mm for reinforcement which is either above the precast units or parallel to the precast units. For reinforcement which crosses infills, with a width greater than 300 mm, the spacing shall be equal to or less than 200 mm and the reinforcement shall be fully developed on each side of the infill. A2 A2

9.3.8.4 *Maximum diameter of longitudinal beam bar in internal beam column joint zones*

For nominally ductile structures the maximum diameter of longitudinal beam bars passing through beam column joint zones shall not exceed the appropriate requirement given below for internal beam column joints:

(a) Where the critical load combination for flexure in a beam at the a face of an internal column includes earthquake actions the ratio of bar diameter to column depth, d_b/h_c, shall not exceed:

$$\frac{d_b}{h_c} = 4\alpha_f \frac{\sqrt{f_c'}}{f_y} \quad \text{.....(Eq. 9–2)}$$

where α_f is taken as 0.85 where the beam bar passes through a joint in a two-way frame and as 1.0 for a joint in a one-way frame.

(b) Where the critical load combination for flexure in a beam at the face of a column either, does not include earthquake actions, or, plastic regions cannot develop adjacent to the face of the column, the ratio of bar diameter to column depth shall not exceed:

$$\frac{d_b}{h_c} = 6\alpha_f \frac{\sqrt{f_c'}}{\left(f_y\left(1+\frac{f_s'}{f_y}\right)\right)} \quad \text{.....(Eq. 9–3)}$$ A2

The value of f_s' is the compression stress in the bar on one side of the joint zone, but need not be taken as greater than $0.5f_y$, and α_f is as defined in (a) above.

A2 **9.3.8.5** *Anchorage of beam bars using hooks in beam column joints*

The bars shall be hooked and satisfy the requirements of 8.6.10, with the hooked end being bent towards the mid-height of the beam and the hook being located as close as possible to the face of the column furthest from the critical section, which is to be taken at the entry point of the bar into the column. The distance between outside edge of the hook and the point of entry into the column shall in all cases be equal to or greater than ¾ h_c.

9.3.9 *Transverse reinforcement in beams and one-way slabs*

9.3.9.1 *General*

Transverse reinforcement shall be the maximum area required for shear combined with torsion or for control of bar buckling.

9.3.9.2 *Diameter and yield strength of transverse reinforcement*

Stirrup or tie reinforcement shall be at least 5 mm in diameter and the design yield strength shall not be taken greater than 500 MPa.

9.3.9.3 *Design for shear*

9.3.9.3.1 *Design shear force adjacent to supports*

A2 The design shear force in a beam at a support may be computed at a critical section a distance of d out from the edge of the support for reinforced concrete and at a distance of $h/2$ out for prestressed concrete, provided the conditions below are satisfied:

(a) The support reaction applies compression to the bottom surface of the beam;
(b) The loads are applied to the top or near the top of the beam; and
(c) No significant concentrated load occurs between the critical section and the support.

9.3.9.3.2 *Design of shear reinforcement*

The design of shear reinforcement shall be based on the assumptions given in 7.5 and be in accordance with 9.3.9.4.

9.3.9.3.3 *Maximum nominal shear stress and effective shear area*

The maximum nominal shear stress, v_n, shall be equal to or less than $0.2f_c'$ or 8 MPa as given by 7.5.

The value of A_{cv} shall:

(a) For rectangular, T- and I- section shapes be taken as product of the web (b_w) width times the effective depth, (d);
(b) For octagonal, circular, elliptical, and similar shaped section A_{cv} shall be taken as the area enclosed by the transverse reinforcement.

9.3.9.3.4 *Nominal shear strength provided by the concrete for normal density concrete, V_c*

The nominal shear strength resisted by concrete, V_c, shall be taken as:

$$V_c = v_c A_{cv} \quad \text{(Eq. 9–4)}$$

where v_c is the shear resisted by concrete.

The value of v_c is given by:

$$v_c = k_d k_a v_b \quad \text{(Eq. 9–5)}$$

A2 where v_b is equal to the smaller of $(0.07+10p_w)\sqrt{f_c'}$ or $0.2\sqrt{f_c'}$, but need not be taken as less than $0.08\sqrt{f_c'}$.

In the calculation for v_b the value of f_c' shall not be taken as greater than 50 MPa.

The factor k_a, in Equation 9–5, allows for the influence of maximum aggregate size on the shear strength. For concrete with a maximum aggregate size of 20 mm or more k_a shall be taken as 1.0. For concrete where the maximum aggregate size is of 10 mm or less, the value of k_a shall be taken as 0.85. Interpolation may be used between these limits.

The factor k_d allows for the influence of member depth on strength and it shall be calculated from any one of the appropriate conditions listed below:

(a) For members with shear reinforcement equal to or greater than the nominal shear reinforcement given in 9.3.9.4.15, k_d = 1.0;
(b) For members with an effective depth equal to or smaller than 400 mm, k_d = 1.0;
(c) For members with an effective depth greater than 400, $k_d = (400/d)^{0.25}$ where d is in mm);
(d) For members with longitudinal reinforcement in the web, with a ratio of 0.003 or more, for the area between the principal flexural tension reinforcement and the mid depth of the beam, and with a bar spacing which does not exceed 300 mm in any direction, k_d, is given by $k_d = (400/d)^{0.25}$, but with limits of $0.9 \le k_d \le 1.0$.

For members with an effective depth of 200 mm or less, the value of v_c shall be taken as the larger of $0.17\, k_a \sqrt{f_c'}$ or the value given by Equation 9–5. For members with an effective depth between 200 mm and 400 mm, the value of v_c shall be found by linear interpolation. A2

9.3.9.3.5 *Nominal shear strength provided by the concrete for lightweight concrete*

Provisions for the nominal shear strength provided by the concrete, apply to normal density concrete. Where lightweight aggregate concrete is used one of the following modifications shall apply:

(a) Where f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, provisions for v_b (in Equation 9–5 shall be modified by substituting 1.8 f_{ct} for $\sqrt{f_c'}$ but the value of 1.8 f_{ct} shall not exceed $\sqrt{f_c'}$;
(b) Where f_{ct} is not specified, all values of $\sqrt{f_c'}$ affecting v_b shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be applied when partial sand replacement is used.

9.3.9.3.6 *Nominal shear strength provided by shear reinforcement*

In accordance with 7.5 the shear reinforcement shall be computed using:

$$V_s = \frac{V^*}{\phi} - V_c \quad \text{.....(Eq. 9–6)}$$

where V_c is the nominal shear strength provided by the concrete given in 9.3.9.3.4 and 9.3.9.3.5 and V_s is the shear strength provided by the shear reinforcement given in 9.3.9.4.

9.3.9.4 *Design of shear reinforcement in beams*

9.3.9.4.1 General

For beams a truss analogy shall be used to determine the nominal shear strength of members with web reinforcement. Either the strut and tie method may be used, in which case V_c shall be taken as zero, or V_c shall be calculated from 9.3.9.3.4 and 9.3.9.3.5 and V_s shall be calculated from 9.3.9.4.2 to 9.3.9.4.8. In either case the requirements of 9.3.9.4.5 to 9.3.9.4.8 shall be satisfied.

9.3.9.4.2 Shear reinforcement perpendicular to longitudinal axis of the beams

When shear reinforcement perpendicular to the longitudinal axis of beams is used and the applied shear is parallel to the legs of rectangular stirrups or ties:

$$V_s = A_v f_{yt} \frac{d}{s} \quad \text{.....(Eq. 9–7)}$$

where A_v is the area of shear reinforcement within distance s.

9.3.9.4.3 Bent-up bars or inclined stirrups of beams

When bent-up bars or inclined stirrups are used as shear reinforcement in beams:

$$V_s = \frac{A_v f_{yt} (\sin\alpha + \cos\alpha) d}{s} \quad \text{.....(Eq. 9–8)}$$

where α is the angle between the bent-up bars or the inclined stirrups and longitudinal axis of the beam.

9.3.9.4.4 Single bar or single group of parallel bars

When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support:

$V_s = A_v f_{yt} \sin\alpha$ .....(Eq. 9–9)

but not greater than $0.25\sqrt{f_c'}\ b_w d$.

9.3.9.4.5 *Series or groups of parallel bent-up bars*

When shear reinforcement in beams consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, the required area shall be equal to or greater than that computed using Equation 9–8.

9.3.9.4.6 *Effective inclined portion of bent-up bar*

The centre three-quarters only of the inclined portion of any longitudinal bent-up bar in a beam shall be considered effective for shear reinforcement.

9.3.9.4.7 *More than one type of shear reinforcement*

Where more than one type of shear reinforcement is used to reinforce the same portion of the web of a beam the shear strength, V_s, shall be computed as the sum of the V_s values computed for the various types.

9.3.9.4.8 *Angle of shear reinforcement not parallel to applied shear*

In members, such as circular or elliptical members, where the angle made by the shear reinforcement intersecting a potential diagonal tension crack varies in direction, only the component of the shear reinforcement which is parallel to the shear force shall be included.

9.3.9.4.9 *Stirrups required where beam frames monolithically into side of girder*

Where a beam of depth h_b and width b, frames monolithically into a supporting girder of depth h_g, stirrups shall be provided in the supporting girder as follows:

(a) The design strength of the stirrups, $\phi\Sigma A_v f_{yt}$ shall equal or be greater than the total reaction transferred from the beams

(b) The stirrups specified in (a) shall be provided within a length of $b + (h_g - h_b)$ about the centreline of the beam.

(c) The requirements of (a) and (b) are waived if the reaction from the beam is introduced within the compression zone of the girder, or the girder is supported below the beam girder joint.

9.3.9.4.10 *Stirrups required for non-monolithic beam-girder connections*

Where a beam frames into a supporting girder, and a monolithic connection is not provided, stirrups shall be provided at the end of the beam with a design strength, $\phi\Sigma A_v f_{yt}$, equal or greater than the total reaction transferred from the beam. These requirements are waived if the beam is supported by bearing on a seating and stirrups in the girder comply with 9.3.9.4.9.

9.3.9.4.11 *Location and anchorage of shear reinforcement*

Stirrups and other bars or wires used as shear reinforcement shall be anchored as required by 7.5.7.

9.3.9.4.12 *Spacing limits for shear reinforcement*

Spacing limits for shear reinforcement shall be as follows:

(a) Spacing of shear reinforcement measured along the axis of the member, shall be equal to or smaller than the smaller of 0.5*d* or 600 mm;

(b) Where the width of the web exceeds 0.5*d* the spacing between stirrup legs measured at right angles to the longitudinal axis the beam shall be equal to or smaller than 0.5*d* or 600 mm but need not be less than 250 mm; A2

(c) Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45° line, extending towards the reaction from mid-depth of member (i.e. 0.5*d*) to the longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement;

(d) When V_s exceeds $0.33\sqrt{f'_c}\, b_w d$, the maximum spacings given in 9.3.9.4.12(a) and (b) shall be reduced by one-half, except in (b) the spacing need not be less than 200 mm.

9.3.9.4.13 *Minimum area of shear reinforcement*

A minimum area of shear reinforcement shall be provided in all reinforced concrete members as required by 7.5.10 when the design shear force exceeds one half of the design shear strength provided by concrete, ϕV_c, except: A2

(a) In beams with a total depth equal to or less than 250 mm;

(b) In beams cast compositely with slabs, where the overall depth is equal to or less than the smaller of half the width of the web, or 300 mm;

(c) In slabs, including floor slabs containing precast pretensioned units, where the maximum clear spacing between the webs is equal to or less than 750 mm; and the overall depth is less than 400 mm;

(d) In monolithic rib and slab construction where the ribs have a width equal to or greater than 100 mm, the clear spacing between ribs is equal to or less than 750 mm and the total depth of the rib and slab is equal to or less than 300 mm;

(e) In cases where tests have shown that satisfactory performance can be achieved when allowance has been made for likely actions including creep, shrinkage, differential temperature, gravity loads, seismic actions and repetitive loading where appropriate. The tests shall be part of a special study, which satisfies the requirements of AS/NZS 1170.5 Appendices A and B.

The exceptions (a), (b), (c) and (d) should not be used when highly repetitive loads occur inducing a shear force due to the variable component of the load exceeding $V_c/3$, or where either slabs are free to translate horizontally at their boundaries or load sharing possibilities in slabs, or between adjacent webs, do not exist.

9.3.9.4.14 *Minimum shear reinforcement waived by testing*

Minimum shear reinforcement requirements of 9.3.9.4.13 may be waived if shown by full scale testing that the required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

9.3.9.4.15 *Minimum area of shear reinforcement*

Where shear reinforcement is required by 9.3.9.4.13, and where 7.6.1.2 allows torsion to be neglected, the minimum area of shear reinforcement for non-prestressed members shall be computed by:

$$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w s}{f_{yt}} \qquad \text{(Eq. 9–10)}$$

9.3.9.5 *Torsional reinforcement*

Except for slabs and footings which are exempted from requirements for torsional reinforcement by 7.6.1.1, torsional reinforcement shall be provided in accordance with 7.6.

9.3.9.6 *Design of transverse reinforcement for lateral restraint of longitudinal bars*

9.3.9.6.1 *Extent of transverse reinforcement*

Stirrups or ties conforming to 9.3.9.6.2 and 9.3.9.6.3 shall be present throughout the length of a beam or slab where longitudinal compression reinforcement is required.

9.3.9.6.2 *Centre-to-centre spacing of transverse reinforcement*

Centre-to-centre spacing of stirrups or ties along the member shall not exceed the smaller of the least lateral dimension of the cross section of the member or 16 longitudinal bar diameters.

9.3.9.6.3 *Arrangement of stirrups or ties*

Stirrups or ties shall be arranged so that every corner and alternate longitudinal bar that is required to function as compression reinforcement shall have lateral support provided by the corner of a stirrup or tie. This lateral support shall be provided by an included angle of not more than 135° and no longitudinal bar shall be further than 150 mm clear on each side from a laterally supported bar. The requirements of 7.5.7.1 shall be satisfied.

9.3.9.6.4 *Enclosure of compression reinforcement*

Stirrup or tie reinforcement shall enclose the longitudinal compression reinforcement in the webs of beams.

9.3.10 *Special provisions for deep beams*

9.3.10.1 *General*

A2 The provisions of 9.3.10 apply to members which satisfy the requirements of 9.3.1.6.1.

9.3.10.2 *Design methods*

Deep beams shall be designed using strut-and-tie models or by taking into account the non-linear distribution of strains. The minimum reinforcement in deep beams is to comply with 9.3.10.3 and 9.3.10.4.

9.3.10.3 *Minimum vertical shear reinforcement*

The area of shear reinforcement perpendicular to the span A_v, shall be equal to or greater than $0.0025 b_w s$,
A2 and s shall not exceed $d/5$, nor 300 mm, but where d is less than 750 mm, s may be taken as 150 mm.

9.3.10.4 *Minimum horizontal shear reinforcement*

The area of shear reinforcement parallel to the span, A_{vh}, shall be equal to or greater than $0.0015 b_w s_2$, and
A2 s_2 shall not exceed $d/5$, nor 300 mm, but where d is less than 750 mm, s_2 may be taken as 150 mm.

9.3.11 *Openings in the web*

9.3.11.1 *General*

Adjacent openings for services in the web of flexural members shall be arranged so that potential failure planes across such openings cannot occur.

9.3.11.2 *Location and size of openings*

Small square or circular openings may be placed in the mid-depth of the web provided that cover requirements to longitudinal and transverse reinforcement are satisfied, and the clear distance between such openings, measured along the member, is equal to or greater than 150 mm. The size of small openings shall not exceed 1000 mm^2 for members with an effective depth less than or equal to 500 mm, or $0.004d^2$ when the effective depth is more than 500 mm.

9.3.11.3 *Larger openings*

Webs with openings larger than that permitted by 9.3.11.2 shall be subject to rational design to ensure that the forces and moments are adequately transferred at and in the vicinity of the openings.

9.3.11.4 *Location and size of large openings*

Whenever the largest dimension of an opening exceeds one-quarter of the effective depth of the member it is to be considered large. Such openings shall not be placed in the web where they could affect the flexural or shear capacity of the member, or where the design shear force exceeds $0.4\sqrt{f_c'}\, b_w d$, or closer

than 1.5*h* to the critical section of a plastic region. In no case shall the height of the opening exceed 0.4*d* or its edge be closer than 0.33*d* to the compression face of the member.

9.3.11.5 *Reinforcement in chords adjacent to openings*

For openings defined by 9.3.11.4, longitudinal and transverse reinforcement shall be placed in the chords at both sides of the opening to resist $1^1/_2$ times the shear force and bending moment generated by the shear across the opening. Shear resistance shall be assigned to each chord in proportion of its stiffness taking into account the effects of cracking and axial compression and tension induced in the chords by the primary moment at the opening.

9.3.11.6 *Reinforcement in webs adjacent to openings*

Transverse web reinforcement, extending over the full depth of the web, shall be placed adjacent to both sides of a large opening over a distance not exceeding one-half of the effective depth of the member to resist twice the entire design shear force across the opening.

9.4 Additional design requirements for members designed for ductility in earthquakes

9.4.1 *Dimensions of beams*

9.4.1.1 *General*

For beams which sustain plastic regions in the ultimate limit state either an analysis based on first principles shall be made to demonstrate that the beam is stable or the dimension limits given in 9.4.1.2, 9.4.1.3 or 9.4.1.4 as appropriate shall be satisfied.

9.4.1.2 *Beams with rectangular cross sections*

The depth, width and clear length between the faces of supports of members with rectangular cross sections, to which moments are applied at both ends by adjacent beams, columns or both, shall be such that:

$$\frac{L_n}{b_w} \le 25 \qquad \text{(Eq. 9–11)}$$

and

$$\frac{L_n h}{b_w^2} \le 100 \qquad \text{(Eq. 9–12)}$$

9.4.1.3 *Cantilevered beams*

The depth, width and clear length from the face of support of cantilever members with rectangular cross sections, shall be such that:

$$\frac{L_n}{b_w} \le 15 \qquad \text{(Eq. 9–13)}$$

and

$$\frac{L_n h}{b_w^2} \le 60 \qquad \text{(Eq. 9–14)}$$

9.4.1.4 *T - and L - beams*

The width of web of T- and L- beams, in which the flange or flanges are integrally built with the web, shall be such that the values given by Equations 9–11 and 9–13 are not exceeded by more than 50 %.

9.4.1.5 *Width of compression face of members*

The width of the compression face of a member with rectangular, T-, L- or I- section shall be equal to or greater than 200 mm.

9.4.1.6 *Slab width effective in tension in negative moment regions of beams*

9.4.1.6.1 *Contribution of slab reinforcement to design strength of beams*

In T- and L- beams built integrally with slabs, slab reinforcement contained within the effective overhanging flange may be considered to contribute to the design flexural strength in ductile and limited ductile plastic regions in beams, as detailed for nominally ductile beams in 9.3.1.4 with the following modification. The 15 percent limit on the proportion of longitudinal reinforcement in an outstanding flange, which may be considered to contribute to flexural strength in 9.3.1.4 (b) (i), is reduced to 10 percent.

9.4.1.6.2 *Contribution of slab reinforcement to overstrength of plastic region in a beam*

In T- and L-beams built integrally with slabs, slab reinforcement in the over-hanging portion of flanges, A2 which are identified in (a), (b), (c) or (e) below, shall be assumed to contribute to the overstrength moment of resistance at the critical section of plastic regions in the beam being considered. Where precast units are contained in a portion of slab within the effective overhanging flange width their contribution to A2 strength shall be included as specified in (d), (e) and (f). In no case need the flexural tension force contribution of an overhanging flange exceed the value given in (g).

(a) Where a beam containing the potential plastic region is at right angles to the edge of the floor and it frames into an exterior column, but no transverse edge beam is present, the effective width of overhanging flange, b_f, shall be taken as the smaller of the distance at the critical section of the potential plastic region in the beam between the web and a line drawn at 45° from the intersection of a line A2 drawn parallel to the web and touching the side of the column and the edge of the slab. Any reinforcement passing through this section shall be assumed to be stressed to 1.1 $\phi_{o,fy}$ f_y, where the value of $\phi_{o,fy}$ is given in 2.6.5.5.

(b) Where a beam containing the potential plastic region is at right angles to the edge of a slab frames into an external column and the slab is supported by a transverse beam, the effective overhanging A2 flange width, b_f, shall be taken as the smaller of the width defined in (a) above plus twice the width of the web of the transverse beam.
The tension force sustained by the overhanging flange shall be calculated as in (a).

(c) Where a beam containing a potential plastic region or regions passes through a column the effective overhanging flange width on each side of the beam shall be taken as the smaller of:
   (i) Three times the overall depth of the beam;
   (ii) The clear distance between adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1}+h_{b2}}\right)$

   Where h_{b1} is the depth of the beam being considered and h_{b2} is the depth of the adjacent beam.

(d) Where precast prestressed components, which are:
   (i) Parallel or near parallel to the beam containing the potential plastic region
   (ii) Span past these potential plastic regions, and
   (iii) Are located within the effective overhanging flange width defined in (c) above, their contribution to the flexural overstrength of the plastic regions shall be calculated.

The contribution of an overhanging flange containing prestressed units consists of two parts:

(i) The tension force sustained by the reinforcement in the *in situ* concrete including the concrete topping above the precast units. This component, T_{tc}, shall be taken as the total area of the non-prestressed reinforcement within the overhanging flange, which is parallel to the web of the beam containing the plastic region, times 1.1 f_y.

(ii) The tension force, which acts at the mid-depth of the topping concrete, that can be sustained by the precast units located within the overhanging flange width. The determination of this force shall either be based on a rational analysis, or on the simplifying assumption that in the limiting condition the compression force in the precast unit is coincident with the prestressing force. With this assumption the tension force resisted by the precast units, T_p, is given by:

$$T_p = \frac{M_f}{e} \quad \text{.....(Eq. 9–15)}$$

Where

e = the distance between the mid-depth of the topping concrete to the centroid of the prestressing force.

M_f = the bending in the effective over-hanging flange located in the plane containing the critical section of the plastic region being considered. This bending moment, M_f, is calculated, as detailed below, assuming precast units and topping concrete comprising this overhanging flange are supported at the ends of the precast units by transverse beams.

The bending moment, M_f, shall be calculated by summing the components due to:

(i) The total dead load of the overhanging flange and the associated long-term live load;

(ii) The positive flexural moments which can be transmitted to the precast units at their supports by reinforcement connecting the units to the transverse beams, with the smaller of the top or bottom reinforcement stressed to $\phi_{o,fy} f_y$;

(iii) The vertical shear forces, which can be transferred between the web of the beam and the first prestressed unit in the flange and between the face of any column located close to the potential plastic regions and the first precast unit. In both cases the shear forces per unit length, v_p, are calculated from the flexural resistance provided by the slab linking the beam web or column to the first precast unit slab by the equation:

$$v_p = \left(\frac{m_{\ell,w} + m_{\ell.p}}{L_l} \right) \quad \text{.....(Eq. 9–16)}$$

Where:

$m_{\ell,w}$ = the flexural strength of the linking slab per unit length at the face of the web or column;

$m_{\ell,p}$ = the flexural strength of the linking slab at the face of the first precast unit;

L_ℓ = the span of the linking slab, between the face of the web and the face of the first precast unit, or between the face of the column and first precast unit.

The flexural strengths of the linking slab per unit length, $m_{l,w}$ and $m_{l,p}$ shall be based on standard flexural ultimate strength theory but assuming the stress in the reinforcement is $1.1 f_y$ and a rectangular concrete stress block with a stress of $0.2 f_c'$, where the span is between the web of the beam and the first precast unit and $0.8 f_c'$ where the span is between the column face and first precast unit. A2

(e) Where a beam containing a potential plastic region or regions passes through a column with a transverse beam framing into it, the effective flange width on the side or sides with the transverse shall be taken as:

(i) The smaller of the clear distance between the adjacent beams times the factor $\left(\frac{h_{b1}}{h_{b1} + h_{b2}} \right) \leq 4 h_{b1}$ for the case where the transverse beams support precast floor units;

(ii) For the case where transverse beam does not support precast floor units, the effective flange width is equal to the value given above, but the limit of $4h_{b1}$ replaced by $3h_{b1}$.
Where h_{b1} is the depth of the beam being considered and h_{b2} is the width of the adjacent beam.

The tension force contribution of the flange shall be taken as the area of reinforcement connecting the flange to the transverse beam times a stress in this reinforcement of $1.1\ \phi_{o,fy} f_y$.

(f) For situations which are not covered by (a), (b) (c) (d) or (e), the contribution of the flange shall be based on a rational extension of the effective flange widths and method of calculation in (a), (b), (c), (d) and (e).

(g) The contribution of a flange to the overstrength tension force, T_f, in a beam need not be taken greater than:

$T_f = \phi_{o,fy} f_y A_\ell + 2.0 f_y A_t$ .....(Eq. 9–17)

Where:

A_ℓ is the area of longitudinal reinforcement, (parallel to web) in the overhanging flange;

f_y is the design yield stress;

$\phi_{o,fy}$ is the overstrength factor for the reinforcement given in 2.6.5.6;

A_t is the area of reinforcement transverse to the web in the slab which lies within the distance x;

X is the distance to the end of the prestressed unit from the critical section of the plastic region being considered;

The value of T_f at the section being considered may be taken as the smaller of the values calculated from each end of the precast unit.

In calculating the flexural overstrength in a plastic region, when the flexural compression force is on the flange side of the member, the effective width of slab on each side of the beam, which contributes to the resistance of the compression force, shall be taken as equal to 4 times the thickness of the slab adjacent to the web.

9.4.1.6.3 *Diameter and extent of slab bars*

The diameter of bars in that part of the slab specified in 9.4.1.6.1 shall not exceed one-fifth of the slab thickness. Such bars, when subjected to tension, shall extend by the horizontal distance from the position of the bar to the centre of the beam section beyond the point specified in 8.6.12.3.

9.4.1.7 *Narrow beams and wide columns*

Where narrow beams frame into wide columns, the width of a column that shall be assumed to resist the forces transmitted by the beam shall be in accordance with 15.4.6.

9.4.1.8 *Wide beams at columns*

Where wide beams frame into columns the width of beam that shall be assumed to resist the forces transmitted by the column shall be no more than the width of the column plus a distance on each side of the column equal to one-quarter of the overall depth of the column in the relevant direction.

9.4.2 *Potential yielding regions*

A2 Special detailing is required in ductile and limited ductile plastic regions, extending from the critical section of the plastic regions over a length equal to the ductile detailing length, ℓ_y.

These regions and lengths shall be located as follows:

(a) Where the critical section is located at the face of a supporting column, wall or beam: over a length equal to twice the beam depth, measured from the critical section toward mid-span, at each end of the beam where a plastic region may develop;

(b) Where the critical section is located at a distance equal to or greater than either the beam depth h or 500 mm away from a column or wall face: over a length that commences between the column or wall face and the critical section, at least either $0.5h$ or 250 mm from the critical section, and extends at least $1.5h$ past the critical section toward mid-span;

(c) Where, within the span, yielding of longitudinal reinforcement may occur only in one face of the beam as a result of inelastic displacements of the frame: over the lengths equal to twice the beam depth on both sides of the critical section.

9.4.3 *Longitudinal reinforcement in beams containing ductile or limited ductile plastic regions*

9.4.3.1 *Development of beam reinforcement*

The distribution and curtailment of the longitudinal beam reinforcement shall be such that the flexural overstrength of a section can be attained at critical sections in potential plastic hinge regions.

9.4.3.2 *Anchorage of beam bars in columns or beam studs*

9.4.3.2.1 *Point of commencement of bar anchorage*

When longitudinal beam bars are anchored in cores of exterior and interior columns or beam stubs, the anchorage for tension shall be deemed to commence at one-half of the relevant depth of the column or 8 d_b, whichever is less, from the face at which the beam bar enters the column. Where it can be shown that the critical section of the plastic hinge is at a distance of at least the beam depth or 500 mm, whichever is less, from the column face, the development length may be considered to commence at the column face of entry.

9.4.3.2.2 *Reinforcement of beam stubs*

Where longitudinal beam bars are terminated in beam stubs, the development length of the bars in compression shall be assumed to commence at the smaller of $8d_b$ or $h/2$ from the face of the column where the bar enters the joint zone. Where the development length in compression is inadequate to anchor the bar, reinforcement shall be provided to constrain the vertical leg of the bar. Reinforcement providing this constraint shall have a tension capacity equal to or greater than one twelfth of the yield force in the bar or bars to be anchored, and it shall be located within a vertical distance of 8 times the diameter of the anchored bar or bars measured from the longitudinal axis of the bar or bars. A2

9.4.3.2.3 *Development length*

For calculation of the development length, the reduction provisions of 8.6.3.3, 8.6.8.2 and 8.6.10.3 by $\alpha_b = A_{sr}/A_{sp}$ shall not apply.

9.4.3.2.4 *Anchorage of diagonal bars in coupling beams*

When three or more diagonal or horizontal bars of a coupling beam are anchored in adjacent structural walls, the development length shall be 1.5 times the development length computed from 8.6.3 and 8.9.2. A2

9.4.3.2.5 *Bars to terminate with a hook or anchorage device*

Notwithstanding the adequacy of the anchorage of a beam bar in a column core or a beam stub, no bar shall be terminated without a vertical 90° standard hook or equivalent anchorage device as near as practically possible to the far side of the column core, or the end of the beam stub where appropriate, and not closer than three-quarters of the relevant depth of the column to the face of entry. Top beam bars shall only be bent down and bottom bars must be bent up.

9.4.3.3 *Maximum longitudinal reinforcement in beams containing ductile plastic regions*

At any section of a beam within a ductile detailing length, ℓ_y, as defined in 9.4.2, the tension reinforcement ratio, p, shall not exceed:

$$p_{max} = \frac{f_c' + 10}{6f_y} \le 0.025 \quad \text{(Eq. 9–18)}$$

where the reinforcement ratio, p, shall be computed using the width of the web.

9.4.3.4 *Minimum longitudinal reinforcement in beams containing ductile plastic regions*

When determining the longitudinal reinforcement in beams of ductile structures:

(a) Any section within the ductile detailing length, ℓ_y, as defined in 9.4.2, shall have an area of compression reinforcement, A_s', which is restrained against buckling as required by 9.4.5, equal to or greater than the appropriate proportion of flexural tension reinforcement, A_s, given below: A2

(i) $A_s' > 0.5A_s$ for ductile plastic regions defined in 2.6.1.3

(ii) $A_s' > 0.38A_s$ for limited ductile plastic regions defined in 2.6.1.3

The area A_s is the area of flexural tension reinforcement provided within the section defined in 9.4.1.6.1 to meet the ultimate strength requirements.

This requirement need not be complied with when the compression reinforcement is placed within the depth of a compression flange of a T- or L- beam formed as a cast-in-place concrete floor slab built integrally with the web at a section subjected to positive bending moment, or where a uni-directional

positive moment plastic hinge forms in the span of a beam supporting precast concrete floor units which have a cast-in-place concrete topping of thickness 60 mm or more.

(b) At any section of a beam the minimum longitudinal reinforcement ratio, p, for both top and bottom reinforcement computed using the width of the web shall exceed that given by:

$$p_{min} = \frac{\sqrt{f_c'}}{4f_y} \quad \text{(Eq. 9–19)}$$

(c) At least one quarter of the larger of the top flexural reinforcement required at either end of a beam shall be continued throughout its length. At least two 16 mm diameter bars shall be provided in both the top and bottom throughout the length of the beam.

9.4.3.5 *Maximum diameter of longitudinal beam bars passing through interior joints of ductile structures*

9.4.3.5.1 *General*

The maximum diameter of Grades 300 and 500 longitudinal beam bars passing through an interior joint
A2 | shall be computed from either 9.4.3.5.2 or 9.4.3.5.3 below provided one of the conditions, (a) to (d), given below is satisfied:

(a) Grade 300 reinforcement is used;

(b) The inter-storey deflections divided by the storey height at the ultimate limit state does not exceed 1.8 % when calculated using the equivalent static or modal response spectrum methods;

(c) The beam column joint zone is protected from plastic hinge formation at the faces of the column (as illustrated in Figure C9.19);

(d) The plastic hinge rotation at either face of the column does not exceed 0.016 radians.

If none of these conditions is satisfied the permissible diameter of Grade 500 beam reinforcement passing through an interior joint shall be determined by multiplying the diameter given by 9.4.3.5.2 or 9.4.3.5.3 below by γ, where:

$\gamma = (1.53 - 0.29\delta_c)$, but not greater than 1.0 ..... (Eq. 9–20)

where

δ_c is the inter-storey drift to inter-storey height expressed as a percentage calculated in accordance with NZS 1170.5.

9.4.3.5.2 *Basic ratio of maximum longitudinal beam bar diameter to column depth*

A2 | For beam bars passing through a column at a beam column joint, the ratio of maximum longitudinal bar diameter to column depth shall comply with the appropriate value given in (a) or (b) below:

(a) Where potential plastic regions exist at the column faces:

$$\frac{d_b}{h_c} \le 3.3\alpha_f\alpha_d \frac{\sqrt{f_c'}}{1.25f_y} \quad \text{(Eq. 9–21)}$$

The value of f_c' in Equation 9–21 shall not exceed 70 MPa:

(i) When beam bars pass through a joint in two directions, as in two-way frames, $\alpha_f = 0.85$. For beam bars in one-way frames, $\alpha_f = 1.0$

(ii) When the potential plastic hinges are classed as:

(A) Ductile plastic regions $\alpha_d = 1.0$

(B) Limited ductile plastic regions $\alpha_d = 1.2$.

(b) When the beam potential plastic hinges are located at a distance of at least the smaller of h or 500 mm away from the column faces so that the beam reinforcement remains in the elastic range on each side of the joint zone, the requirements of 9.3.8.4 shall be satisfied.

9.4.3.5.3 *Alternative ratio of maximum longitudinal beam bar diameter to column depth*

Alternatively by considering additional parameters, the ratio of maximum longitudinal beam bar diameter to column depth may be determined by:

$$\frac{d_b}{h_c} \le 6\left(\frac{\alpha_t \alpha_p}{\alpha_s}\right)\alpha_t \frac{\sqrt{f_c'}}{1.25 f_y} \quad \text{.....(Eq. 9–22)}$$ A2

where the variables are defined as follows:

(a) Values of α_t and α_o are as in 9.4.3.5.2;

(b) $\alpha_t = 0.85$ for a top beam bar where more than 300 mm of fresh concrete is cast below the bar
$\alpha_t = 1.0$ for all other cases

(c) To allow for the beneficial effect of compression on a column:

$$\alpha_p = \frac{N_o}{2 f_c' A_g} + 0.95 \quad \text{.....(Eq. 9–23)}$$

with the limitation of $1.0 \le \alpha_p \le 1.25$.

N_o is the minimum design overstrength axial load determined by capacity design in accordance with appendix D.

(d) The coefficient, α_s, allows for the more severe bond stress conditions at overstrength acting on beam reinforcement passing through a beam column joint, where the strength of the reinforcement in the compression zone is less than the flexural tension force resisted by the beam and tension flanges. The value of α_s is given by: A2

$$\alpha_s = [2.55 - R]\frac{1}{\alpha_d} \quad \text{.....(Eq. 9–24)}$$

where R is the ratio of $\phi_{o,fy} A_s' f_y$ to the flexural tension force sustained by the beam and flanges at overstrength, with the limitation $0.75 \le R \le 1.0$.

9.4.3.6 *Splices in longitudinal reinforcement of beams of ductile structures*

9.4.3.6.1 *General*

Splices in longitudinal reinforcement in beams and one-way slabs shall comply with 8.7 and 8.9.1.

9.4.3.6.2 *Location of splices*

Full strength welded splices meeting the requirements of 8.7.4.1(a) and 8.7.4.2 may be used in any location. For all other splices in beams no portion shall be located in a beam column joint region, or within one effective depth of member from the critical section of a potential plastic region in a beam where stress reversals in spliced bars could occur, unless 8.9.1.2 is complied with.

9.4.4 ***Transverse reinforcement in beams of ductile structures***

9.4.4.1 *Design for shear in beams of ductile structures*

9.4.4.1.1 *Design shear strength*

The design shear at a section in a beam, V_o^* shall be determined from consideration of the flexural overstrength being developed at the most probable location of critical sections within the member or in adjacent members, and the gravity load with load factors as specified in 2.6.5.2.

9.4.4.1.2 *Design of shear reinforcement*

Design of shear reinforcement shall be in accordance with 7.5 and 9.3.9.3.6 but with V_c in potential plastic regions being taken as defined in 9.4.4.1.3.

9.4.4.1.3 *Nominal shear strength provided by concrete in potential plastic hinge regions of beams*

In potential plastic hinge regions defined in 9.4.2 the nominal shear strength provided by the concrete, V_c, shall be taken from the appropriate criterion below:

(a) In potential ductile plastic regions, V_c, shall be taken as zero;

(b) In potential limited ductile plastic regions, V_c, shall be taken as not more than half the value given by 9.3.9.3.4.

A2 **9.4.4.1.4** *Sliding shear in reversing plastic regions*

Ductile and limited ductile reversing plastic regions in beams shall be designed to resist sliding shear on sections normal to the axis of the beam, as detailed below:

(a) The maximum shear force, V_o^*, at the critical section of the plastic region shall be equal to or less than the smaller of $0.16\, f_c' A_{cv}$ or $0.85\, A_{cv} \sqrt{f_c'}$, unless the requirements of 9.4.4.1.7 are satisfied;

(b) Where the value of V_o^* exceeds $0.25\,(2+r)\sqrt{f_c'}\, A_{cv}$, diagonal reinforcement shall be provided to resist a shear force, V_{di}, equal to or greater than:

$$V_{di} \geq 0.7\left[\frac{V_o^*}{A_{cv}\sqrt{f_c^*}} + 0.4\right](-r)V_o^* \leq V_o^* \quad \text{.....(Eq. 9–25)}$$

Whereby taking into account the sense of the reversing total shear forces resulting from the two directions of earthquake actions, V_{di},needs to be considered only where $-1.0 < r < -0.2$.

Where r is the algebraic ratio at the critical section of the plastic hinge of the numerically smaller to the larger shear force when the reversal of direction of shear force can occur, and is always taken as negative;

(c) The diagonal reinforcement required to resist V_{di} shall be taken as the transverse component of the force or forces acting in the diagonal reinforcement when it sustains its design yield stress in tension or compression. This reinforcement may be calculated from the expression:

$$V_{di} = f_y \sin\alpha\,(A_{vd} + A_{vd}')$$

where A_{vd} and A_{vd}' are the areas of diagonal reinforcement in two directions placed at an angle of α to the longitudinal axis of the beam. The diagonal reinforcement may be placed in one or two directions;

(d) Where diagonal reinforcement is required, it shall extend for a distance equal to or greater than the effective depth of the beam from the critical section of the plastic region;

(e) The angle, α, between the diagonal reinforcement and the longitudinal axis of the beam, shall be within the range of 30° to 60°;

(f) Where diagonal reinforcement is assumed to act in compression in contributing to V_{di}, and the reinforcement is bent within the clear span of the beam, stirrups normal to the axis of the beam shall be located within the region of the bend in the diagonal reinforcement to resist the vertical component of the diagonal compression force.

9.4.4.1.5 *Design for conventional shear in plastic hinge*

Design for shear reinforcement to maintain equilibrium across a diagonal tension crack shall be in accordance with 9.4.4.1 and 7.5. The vertical component of the tension force carried by the diagonal reinforcement which crosses the potential diagonal crack may be added to the shear resistance provided by stirrups to give the shear force resisted by web reinforcement.

9.4.4.1.6 *Minimum shear reinforcement*

A2 Stirrups which are anchored round the top and bottom reinforcement in the beam shall be provided over the clear span of the beam. The spacing of stirrups shall be equal to or less than the smaller of $12d_b$ or $d/2$, where d_b is the diameter of the smallest longitudinal bar in the corners of the stirrups near the top and bottom faces of the beam. The area of each stirrup, A_v, shall be equal to or greater than:

$$A_v = \frac{\sqrt{f_c'}}{12}\frac{b_w s}{f_y} \quad \text{.....(Eq. 9–27)}$$ A2

9.4.4.1.7 *Diagonally reinforced coupling beams*

Diagonally reinforced coupling beams shall not be used where L_{nd} / h_b exceeds 4, where h_b is the overall depth of the beam.

(a) Diagonally reinforced coupling beams shall be used where either:
  (i) The shear force exceeds the allowable given in 9.4.4.1.4(a);
  (ii) The seismic induced deformation exceeds the limit that can be sustained by flexural rotation in plastic regions (2.6.1), but is less than the shear deformation limit for diagonally reinforced coupling beams (2.6.1).

(b) In diagonally reinforced coupling beams, the seismic design moment and shear shall be designed to be resisted by two sets of diagonal reinforcement which intersect at the mid-section of the coupling beam. Each of these two sets of reinforcement shall consist of 4 or more bars, and each set shall be separately enclosed by rectangular ties (or equivalent spirals), and satisfy the following:
  (i) Stirrup ties shall be arranged so that each bar within the length, L_{nd}, is restrained against buckling by a 90° bend in a stirrup-tie at a spacing of equal to or less than $6d_b$, except where two or more bars at not more than 200 mm centres are so restrained; any bars between them are exempt from this requirement;
  (ii) Diagonally reinforced coupling beams shall contain longitudinal reinforcement which satisfies 9.4.3.4(b) and stirrups enclosing the top and bottom reinforcement which satisfies Eq. 9–27 (in 9.4.4.1.6), with a spacing that is equal to or less than the smaller of 12 times the diameter of the longitudinal bars or $h_b/4$;
  (iii) The diagonal reinforcement shall be anchored in adjacent members by a length equal to or greater than 1.5 times its development length in tension (L_d).

9.4.5 *Design of transverse reinforcement for lateral restraint of longitudinal bars of beams of ductile structures*

Transverse reinforcement in the form of stirrup-ties shall be placed in potential plastic regions of beams, as defined in 9.4.2 as follows:

(a) Stirrup-ties shall be arranged so that each longitudinal bar or bundle of bars in the upper and lower faces of the beam is restrained against buckling by a 90° bend of a stirrup-tie, except that where two or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement;

(b) The diameter of the stirrup-ties shall be equal to or greater than 5 mm, and the area of one leg of a stirrup-tie in the direction of potential buckling of the longitudinal bar shall be equal to or greater than:

$$A_{te} = \frac{\Sigma A_b f_y}{96 f_{yt}}\frac{s}{d_b} \quad \text{.....(Eq. 9–28)}$$

where s is the spacing of stirrup-ties, ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 9.4.5(a) and f_{yt} shall not be taken larger than 800 MPa. Longitudinal bars centred more than 75 mm from the inner face of stirrup-ties need not be considered in determining the value of ΣA_b;

(c) If a horizontal layer of longitudinal bars is centred further than 100 mm from the inner face of the adjacent horizontal leg of stirrup-ties, the outermost bars shall be tied laterally as required in 9.4.5(b), unless this layer is situated further than $h/4$ from the compression edge of the section;

(d) In potential plastic hinge regions defined by 9.4.2(a) and (b) the centre-to-centre spacing of stirrup-ties for a ductile plastic region (DPR) shall not exceed the smaller of $d/4$ or 6 times the diameter of any longitudinal bar to be restrained in the outer layers. The centre-to-centre spacing in a limited ductile plastic region (LDPR) shall not exceed the smaller of $d/4$ or 10 times the diameter of any longitudinal bar to be restrained in the outer layers. Where 9.4.2(a) applies, the first stirrup-tie in a

beam shall be as close as practicable to the column ties and shall be not further than 50 mm from the column face.

(e) In potential plastic hinge regions defined by 9.4.2(c) the centre-to-centre spacing of stirrup-ties shall not exceed either $d/3$ or ten times the diameter of any longitudinal compression bar to be restrained. The area of stirrup-ties need not satisfy Equation 9–28. When the potential plastic hinge region defined by 9.4.2(c) overlaps that defined by 9.4.2(a) or (b), the spacing and area of stirrup ties shall be governed by the requirements of 9.4.2(a) or (b), respectively.

(f) Stirrup-ties shall be assumed to contribute to the shear strength of the beam.

10 DESIGN OF REINFORCED CONCRETE COLUMNS AND PIERS FOR STRENGTH AND DUCTILITY

10.1 Notation

A_b area of a longitudinal bar, mm^2

A_c area of concrete core of section measured to outside of peripheral spiral or hoop, mm^2

A_{cv} area of concrete assumed to resist shear, (see10.3.10.2.1), mm^2 A2

A_g gross area of section, mm^2

A_h area of hoop or spiral bar at spacing, s, mm

A_{sh} total effective area of hoop bars and supplementary cross-ties in the direction under consideration within spacing s_h, mm^2

A_{st} total area of longitudinal reinforcement, mm^2

A_t area of structural steel shape or pipe, mm^2

A_{te} area of one leg of stirrup-tie, mm^2

A_{tr} smaller of area of transverse reinforcement within a spacing s crossing plane of splitting normal to concrete surface containing extreme tension fibres, or total area of transverse reinforcement normal to the layer of bars within a spacing, s, divided by n, mm^2. If longitudinal bars are enclosed within a spiral or circular hoop reinforcement, $A_{tr} = A_t$ when $n \le 6$.

A_v area of shear reinforcement within a spacing s, mm^2

b width of compression face of member, mm

b_w web width or diameter of circular section, mm

C_m a factor relating actual moment diagram to an equivalent uniform moment diagram

d distance from extreme compression fibre to centroid of tension reinforcement, mm

d'' depth of concrete core of column measured from centre-to-centre of peripheral rectangular hoop, circular hoop or spiral, mm

d_b diameter of reinforcing bar, mm

E_c modulus of elasticity of concrete, MPa, see 5.2.3

E_s modulus of elasticity of steel, MPa, see 5.3.4

EI flexural rigidity of a member. See Equations 10–6 and 10–7 for columns and piers

f_{ct} average split cylinder tensile strength of lightweight aggregate concrete, MPa

f'_c specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement or the yield strength of structural steel casing, MPa

f_{yt} lower characteristic yield strength of spiral, hoop, stirrup-tie or supplementary cross-tie reinforcement, MPa

h overall depth of member, mm

h_b overall depth of beam, mm

h'' dimension of concrete core of rectangular section, measured perpendicular to the direction of the hoop bars, measured to the outside of the peripheral hoop, mm

I_g moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement, mm^4

I_{se} moment of inertia of reinforcement about centroidal axis of member cross section, mm^4

I_t moment of inertia of structural steel shape or pipe about centroidal axis of composite member section, mm^4

k effective length factor for a column or pier

ℓ_p effective length for determining curvatures in a plastic region, mm.

ℓ_y ductile detailing length, mm

L_n clear length of member measured from face of supports, mm

L_u unsupported length of a column or pier, mm

m $f_y/(0.85\, f'_c)$

M_c moment to be used for design of a column or pier, N mm

M_1 value of smaller design end moment on a column or pier calculated by conventional elastic frame analysis, positive if member is bent in single curvature, negative if bent in double curvature, N mm

M_2 value of larger design end moment on a column or pier calculated by conventional elastic frame analysis, always positive, N mm

M^* design moment at section at the ultimate limit state, N mm

N_c critical load, see Equation 10–5, N

$N_{n,max}$ nominal axial load compressive strength of column when the load is applied with zero eccentricity, N

N_o^* design axial load derived from overstrength considerations (capacity design), N

N^* design axial load at ultimate limit state to be taken as positive for compression and negative for tension, N

n number of bars

p_s ratio of volume of spiral or circular hoop reinforcement to total volume of concrete core (outside to outside of spirals or hoops)

p_t ratio of non-prestressed longitudinal column reinforcement = A_{st}/A_g

A2 p_w proportion of flexural tension reinforcement within one-quarter of the effective depth of the member closest to the extreme tension reinforcement to the shear area, A_{cv}. For circular of octagonal columns, p_w may be taken as 0.33 A_{st}/A_{cv}

r radius of gyration of cross section of a column or pier, mm

s centre-to-centre spacing of stirrup-ties along member, mm

s_h centre-to-centre spacing of hoop sets, mm

t_s thickness of steel encasing concrete in a composite member, mm

V shear force, N

v_b shear resisted by concrete in an equivalent reinforced concrete beam, N

V_c nominal shear strength provided by the concrete mechanisms, N

V_E shear force derived from lateral earthquake forces for the ultimate limit state, N

v_n total nominal shear strength of section, N

V_n total nominal shear strength of cross section of column or pier, N

V_s nominal shear strength provided by the shear reinforcement, N

V^* design shear force at the section at the ultimate limit state, N

A2

α_1 factor defined in 7.4.2.7

β_d ratio of design axial dead load to total design axial load of a column or pier

θ angle between the inclined crack and the horizontal axis of column or pier

ΣA_b sum of areas of longitudinal bars, mm^2

δ moment magnification factor, see 10.3.2.3.5

ϕ strength reduction factor, see 2.3.2.2

10.2 Scope

The provisions of this section shall apply to the design of reinforced concrete members for flexure and shear with axial force. The provisions of this and earlier sections are summarised in Table C10.2. The written requirements take precedence over Table C10.2. Columns containing plastic regions with sectional curvature ductility demands less than or equal to the limits for nominally ductile plastic regions defined in Table 2.4 shall meet the requirements of 10.3. Columns containing plastic regions designed for greater sectional ductility demand than this and columns in frames with ductile or limited ductile beam hinges shall meet the requirements of 10.3 as modified by 10.4.

10.3 General principles and design requirements for columns and piers

10.3.1 *Strength calculations at the ultimate limit state*

Columns and piers shall be designed for the most unfavourable combination of design moment, M^*, design axial force, N^*, and design shear force V^*. The maximum design moment, M^*, shall be magnified for slenderness effects in accordance with 10.3.2.

10.3.2 *Slenderness effects in columns and piers*

10.3.2.1 *Design considerations for columns and piers*

The design of columns and piers shall be based on forces and moments determined from a second-order analysis of the structure, or by simplified methods such as described in 10.3.2.2, or alternatively by complying with the section dimension limits given in 10.4.3. Such analysis shall take into account the influence of axial loads and variable moments of inertia due to cracking on member stiffness and end moments, the effect of deflections on moments and forces, and the effects of duration of loads, shrinkage and creep and interaction with the supporting foundations. A2

10.3.2.2 *Evaluation of slenderness effects in columns and piers braced against sidesway*

In lieu of the detailed procedure prescribed in 10.3.2.1, slenderness effects in columns and piers braced against sidesway may be evaluated in accordance with the approximate procedure presented in 10.3.2.3.

10.3.2.3 *Approximate evaluation of slenderness effects*

The approximate method of the evaluation of slenderness effects for columns and piers braced against sidesway given by 10.3.2.3.1 to 10.3.2.3.6, may be used in lieu of that in 10.3.2.1 provided that:

(a) The member cannot form ductile or limited ductile plastic regions in the ultimate limit state;

(b) The relative displacement of the ends of the member δ_o, in the ultimate limit state is such that:

$$N^*\delta_o \le 0.05\, V^* L_u \qquad \text{(Eq. 10–1)}$$

Where L_u is defined in 10.3.2.3.1.

10.3.2.3.1 *Unsupported length*

The unsupported length of a column or pier shall be determined as follows:

(a) The unsupported length, L_u, shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support for that column or pier, in the direction being considered;

(b) Where column capitals or haunches are present, the unsupported length shall be measured to the lower extremity of the capital or haunch in the plane considered.

10.3.2.3.2 *Effective length factor*

The effective length factor, k, of a column or pier braced against sidesway shall be taken as 1.0, unless analysis shows that a lower value may be used.

10.3.2.3.3 *Radius of gyration*

The radius of gyration, r, shall be taken equal to 0.30 times the overall dimension in the direction stability is being considered for a rectangular column or pier, and 0.25 times the diameter for circular column or pier. For other shapes, r shall be computed for the gross concrete section.

10.3.2.3.4 *Consideration of slenderness*

In compression members braced against sidesway, slenderness effects may be ignored for compression members that satisfy:

$$\frac{kL_u}{r} \le 34 - 12(M_1/M_2) \qquad \text{(Eq. 10–2)}$$

where the term [34 – 12 M_1/M_2] shall not be taken greater than 40. The term M_1/M_2 is positive if the member is bent in single curvature, and negative if the member is bent in double curvature.

10.3.2.3.5 *Design actions including slenderness effects*

Design actions including the effects of slenderness shall be determined as follows:

(a) Columns and piers shall be designed using the design ultimate load, N^*, from a first order analysis and a magnified ultimate moment, M_c, defined by:

$$M_c = \delta M_2 \quad \text{(Eq. 10–3)}$$

where

$$\delta = \frac{C_m}{1-\left(\frac{N^*}{0.75N_c}\right)} \geq 1.0 \quad \text{(Eq. 10–4)}$$

and

$$N_c = \frac{\pi^2 EI}{(kL_u)^2} \quad \text{(Eq. 10–5)}$$

In lieu of a more accurate calculation, EI in Equation 10–5 shall be taken either as:

$$EI = \frac{E_c I_g / 5 + E_s I_{se}}{1+\beta_d} \quad \text{(Eq. 10–6)}$$

or conservatively as:

$$EI = \frac{(E_c I_g / 2.5)}{1+\beta_d} \quad \text{(Eq. 10–7)}$$

(b) In Equation 10–4 for members braced against sidesway and without transverse loads between supports, C_m shall be taken as:

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \quad \text{(Eq. 10–8)}$$

where M_1/M_2 is positive if the column is bent in single curvature, and negative if the member is bent in double curvature. For members with transverse loads between supports, C_m shall be taken as 1.0.

In Equations 10–6 to 10–8, β_d shall be taken as the ratio of the maximum design axial dead load (permanent action) to the maximum design axial load.

(c) The moment M_2 in Equation 10–3 shall be taken not less than:

$$M_{2,min} = N^* (15 + 0.03h) \quad \text{(Eq. 10–9)}$$

about each axis separately. For members for which $M_{2,min}$ exceeds M_2, the value of C_m in Equation 10–8 shall either be taken equal to 1.0, or shall be based on the ratio of the computed end moments M_1 and M_2.

10.3.2.3.6 *Bending about both principal axes*

For columns and piers subject to bending about both principal axes, the moment about each axis shall be magnified by δ computed from the corresponding conditions of restraint about that axis.

10.3.3 *Design cross-sectional dimensions for columns and piers*

10.3.3.1 *Compression member with multiple spirals*

Outer limits of the effective cross section of a column or pier with two or more interlocking spirals shall be taken as the distance between the extreme limits of the spirals plus the minimum concrete cover around the peripheral spiral bars required by Sections 3 and 4.

10.3.3.2 *Equivalent circular compression member*

In lieu of using the gross area for design, a column or pier with a square, octagonal, or other shaped cross section may be considered as a circular section with a diameter equal to the least lateral dimension of the actual shape. The required percentage of reinforcement, and design strength shall be based on that circular section.

10.3.4 *Strength of columns and piers in bending with axial force*

10.3.4.1 *General assumptions for flexural and axial force design*

The design of columns and piers for flexure and axial force at the ultimate limit state shall be in accordance with 7.4 and shall be based on satisfaction of applicable conditions of equilibrium and compatibility of strains.

10.3.4.2 *Limit for design axial force, N *, on columns and piers*

For columns and piers the ultimate axial load in compression, N^*, shall be less than $0.85\ \phi N_{n,max}$ for members where:

$$N_{n,max} = \alpha_1 f_c' (A_g - A_{st}) + f_y A_{st} \quad \text{(Eq. 10–10)}$$

where α_1 is given by 7.4.2.7(c).

10.3.5 *Transmission of axial force through floor systems*

10.3.5.1 *Transmission of load through floor system*

When the specified compressive strength of concrete in a column is greater than 1.4 times that specified for a floor system, transmission of load through the floor system shall be as provided for by one of 10.3.5.2, 10.3.5.3 or 10.3.5.4.

10.3.5.2 *Placement of concrete in floor*

Concrete of the strength specified for the column shall be placed in the floor at the column location. The top surface of the column concrete shall extend 600 mm into the slab from the face of the column. The column concrete shall be well integrated into the floor concrete.

10.3.5.3 *Strength of column through floor*

The strength of a column through a floor system shall be based on the lower value of concrete strength. To achieve a column strength through the floor system comparable with the column above and below the slab, supplementary reinforcement in the column, through the floor system, fully developed above and below the slab and adequately confined, may be provided.

10.3.5.4 *Strength of columns laterally supported on four sides*

For columns laterally supported on four sides by beams of approximately equal depth or by slabs, the strength of the column may be based on an assumed concrete strength in the column joint equal to 75 % of the column concrete strength plus 35 % of the floor concrete strength. In the application of this clause, the ratio of column concrete strength to slab concrete strength shall not be taken greater than 2.5 for design.

10.3.6 *Perimeter columns to be tied into floors*

Columns at the perimeter of a floor shall be tied back into the floor by either reinforced concrete beams or tie reinforcement provided in the topping. The tie reinforcement shall be effectively anchored perpendicular to the frame and capable of resisting the larger of 5 % of the maximum total axial A2

A2 compression load acting on the column at the level being considered, or 20 % of the shear force induced by the seismic design actions in the column in the storey below the level being considered.

10.3.7 *Strength of columns and piers in torsion, shear and flexure*

The design of columns and piers for torsion, shear and flexure at the ultimate limit state shall be in accordance with 7.5 and 10.3.1, 10.3.4 and 10.3.10.2.

10.3.8 *Longitudinal reinforcement in columns and piers*

10.3.8.1 *Limits for area of longitudinal reinforcement*

The area of longitudinal reinforcement for columns and piers shall be greater than 0.008, times the gross area, A_g of the section or at any location including lap splices, and less than 0.08 times the gross area, A_g.

10.3.8.2 *Minimum number of longitudinal bars*

A2 The minimum number of longitudinal bars in a column or pier shall be 8, except that this number may be reduced to 4 or 6 where the clear spacing between adjacent bars on the same side of the section is less than 150 mm and the axial load $N^* \leq 0.1 \phi f_c' A_g$.

10.3.8.3 *Spacing of longitudinal reinforcement*

The centre-to-centre spacing of longitudinal bars in a circular column shall be less than or equal to the larger of one-quarter of the diameter of the section, or 200 mm.

In rectangular sections the maximum permissible centre-to-centre spacing of longitudinal bars, which are cross linked across the cross section, shall depend on the ratio of the longer side, h, to the shorter side, b, as set out in (a) and (b) below.

(a) Where the ratio of $h/b < 2.0$ the maximum permissible spacing shall be the larger of $b/3$ or 200mm.

(b) Where the ratio of $h/b > 2.0$ the maximum spacing shall be as for (a) except in the mid regions of the longer side. In the mid region lying between lines drawn at a distance of the larger of b or 1.5 times the depth to the neutral axis from the extreme fibres, the spacing may be increased to the smaller of $h/4$ or 300 mm.

10.3.8.4 *Cranking of longitudinal bars*

Where longitudinal bars are offset, the slope of the inclined portion of the bar with the axis of the column shall be less than or equal to 1 in 6, and the portions of the bar above and below the offset shall be parallel to the face of the column. Adequate horizontal support at the offset bends shall be provided by ties, spirals, other means of restraints or parts of the floor construction. These shall be placed so that the resultant force, providing the horizontal support for the bursting forces, acts through the centre of the bend. The horizontal thrust to be resisted shall be assumed as 1.5 times the horizontal component of the nominal force in the inclined portion of the bar, assumed to be stressed to f_y.

10.3.9 *Splices of longitudinal reinforcement*

10.3.9.1 *General*

Splices in the longitudinal reinforcement of columns and piers shall comply with 8.7.

10.3.9.2 *Offset column faces*

Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate reinforcing bars lapped as required herein.

10.3.9.3 *Laps designed for full yield stress when stress exceeds 0.5* fy.

Where the stress in the longitudinal bars in a column calculated for any loading condition exceeds 0.5 f_y in tension, either lap splices designed for full yield stress in tension shall be used, or full strength welded splices in accordance with 8.7.4.1(a), high strength welded splices, or high strength mechanical connections in accordance with 8.7.4.1(b) and 8.7.5.2 respectively shall be provided.

10.3.10 *Transverse reinforcement in columns and piers*

10.3.10.1 *General*

Transverse reinforcement shall satisfy the requirements of shear, torsion, confinement of concrete, and lateral restraint of longitudinal bars against premature buckling. The maximum area required for shear combined with torsion, confinement, or control of buckling of bars shall be used.

10.3.10.2 *Design for shear*

Design for shear reinforcement shall be in accordance with 7.5. Where the reaction, in the direction of applied shear, introduces compression to the end regions of continuous or cantilever columns, the maximum design shear force V * at the ultimate limit state for sections located at less than distance d from the face of the support may be taken as that computed at distance d from the face of the support.

10.3.10.2.1 *Maximum permissible nominal shear force and effective shear area*

The maximum total nominal shear stress, v_n, shall not exceed $0.2f_c'$ or 8 MPa. The value of the effective shear area, A_{cv} shall:

(a) For rectangular, T– and I section shapes be taken as the product of the web, b_w width times the effective depth, d;

(b) For octagonal, circular, elliptical, and similar shaped section A_{cv} shall be taken as the area enclosed by the transverse reinforcement with the area of outstanding compression or tension flanges being neglected.

10.3.10.2.2 *Method of design for shear*

For columns or piers a truss analogy shall be used to calculate the contribution of shear reinforcement to shear strength. Either:

(a) The strut and tie method may be used, in which case V_c shall be taken as zero; or

(b) V_c shall be calculated from 10.3.10.3 and V_s calculated from 10.3.10.4.

In either case the requirements of 10.3.10.4.3 to 10.3.10.4.4 shall apply.

10.3.10.3 *Shear strength provided by concrete*

10.3.10.3.1 *Nominal shear strength provided by the concrete for normal density concrete*

For normal density concrete V_c shall be taken as not greater than:

$$V_c = k_a k_n v_b A_{cv} \quad \text{(Eq. 10–11)}$$

where:

k_a is equal to 1.0 for maximum aggregate size of 20 mm or more and equal to 0.85 for a maximum aggregate size of 10 mm. Interpolation may be used for intermediate sizes.

v_b is given by

$$v_b = (0.07 + 10\, p_w) \sqrt{f_c'} \quad \text{(Eq. 10–12)}$$

with limits of

$$0.08 \sqrt{f_c'} < v_b < 0.2 \sqrt{f_c'} \quad \text{(Eq. 10–13)}$$

In the calculation of v_b the value of f_c' shall not be taken greater that 50 MPa.

p_w is equal to the effective area of flexural tension reinforcement, which may be taken as the ratio of the area of longitudinal reinforcement in the section lying between the extreme tension reinforcement, and a line located at a distance of one third of the distance between the extreme compression fibre and the extreme tension reinforcement, measured from this reinforcement;

A_{cv} is defined in 10.3.10.2.1;

k_n allows for the influence of axial load and it is given for members subjected to axial compression by:

$$k_n = \left[1 + \frac{3N^*}{A_g f_c'}\right] \quad \text{.....(Eq. 10–14)}$$

for members subjected to axial tension, where N^* is negative for tension is given by:

$$k_n = \left[1 + \frac{12N^*}{A_g f_c'}\right] \geq 0 \quad \text{.....(Eq. 10–15)}$$

10.3.10.3.2 *Change in shear strength in members where sides are not parallel to the longitudinal axis*

A2 In members where the sides are not parallel to the longitudinal axis, allowance shall be made for any decrease in shear resistance due to the transverse component of the compression, and tension forces due to flexure and axial load. The decrease in shear strength is associated with the depth increasing in the direction of decreasing moment.

10.3.10.3.3 *Nominal shear strength provided by the concrete for lightweight concrete*

Provisions for shear strength provided by the concrete, V_c, apply to normal density concrete. Where lightweight aggregate concrete is used one of the following modifications shall apply:

(a) Where f_{ct} is specified and the concrete mix is designed in accordance with NZS 3152, provisions for V_c shall be modified by substituting $1.8f_{ct}$ for $\sqrt{f_c'}$ but the value of $1.8f_{ct}$ shall not exceed $\sqrt{f_c'}$;

(b) Where f_{ct} is not specified, all values of $\sqrt{f_c'}$ affecting V_c shall be multiplied by 0.75 for "all-lightweight" concrete, and 0.85 for "sand-lightweight" concrete. Linear interpolation shall be applied when partial sand replacement is used.

10.3.10.4 *Shear reinforcement*

10.3.10.4.1 *Required nominal shear strength from reinforcement*

In accordance with 7.5, the required shear strength shall be computed using:

$$V_s = \frac{V^*}{\phi} - V_c \quad \text{.....(Eq. 10–16)}$$

where V_c is the nominal shear strength provided by the concrete given in 10.3.10.3 and V_s is the shear strength provided by the shear reinforcement given in 10.3.10.4.2.

10.3.10.4.2 *Nominal shear strength provided by shear reinforcement*

When shear reinforcement perpendicular to the longitudinal axis of columns is used:

(a) for rectangular hoops or ties:

$$V_s = \frac{A_v f_{yt} d}{s} \quad \text{.....(Eq. 10–17)}$$

(b) For circular hoops or spirals:

$$V_s = \frac{\pi}{2}\frac{A_h f_{yt} d''}{s} \quad \text{.....(Eq. 10–18)}$$

where d'' is the depth of the core dimension from centre-to-centre of peripheral hoop or spiral, and where A_h is the area of hoop or spiral bar at spacing, s.

(c) For other sections where the angle the shear reinforcement intersecting a potential diagonal tension crack varies in direction, only the component of the shear reinforcement which is parallel to the shear force shall be included.

10.3.10.4.3 *Maximum spacing of shear reinforcement*

Where the nominal shear resisted by the reinforcement, V_s, exceeds a value of $0.33\sqrt{f_c'}\,A_{cv}$, the spacing shall not exceed $d/4$.

10.3.10.4.4 *Minimum shear strength provided by shear reinforcement*

The area of shear reinforcement shall be equal to or greater than:

$$A_v = \frac{1}{16}\sqrt{f_c'}\,\frac{b_w s}{f_{yt}} \quad \text{.....(Eq. 10–19)}$$

10.3.10.5 *Design of spiral or circular hoop transverse reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.3.10.5.1 *Confinement and anti-buckling reinforcement*

The volumetric ratio, p_s, shall be equal to or greater than that given by the greater of Equation 10–20 or Equation 10–21 for confinement of concrete and lateral restraint of longitudinal bars:

(a) For confinement of concrete:

$$p_s = \frac{(1-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f_c'}{f_{yt}}\frac{N^*}{\phi f_c' A_g} - 0.0084 \quad \text{.....(Eq. 10–20)}$$

where N^* is the maximum design axial load for load combinations involving wind or seismic actions, or any other load case in which significant lateral force is applied to the structure as a whole;

A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions;

The value of $p_t m$ used in the equation shall not be taken greater than 0.4.

where A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions; the value of $p_t m$ used in the equation shall not be taken greater than 0.4.

(b) For lateral restraint of longitudinal bars against premature buckling:

$$p_s = \frac{A_{st}}{155\,d''}\frac{f_y}{f_{yt}}\frac{1}{d_b} \quad \text{.....(Eq. 10–21)}$$

In Equations 10–20 and 10–21, f_{yt} shall not exceed 800 MPa.

10.3.10.5.2 *Spacing of spirals or circular hoops*

The centre-to-centre spacing of spirals or circular hoops along the member shall be less than or equal to the smaller of one-third of the diameter of the cross section of the member or ten longitudinal bar diameters. Clear spacing shall be equal to or greater than 25 mm.

10.3.10.6 *Design of rectangular hoop and tie transverse reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.3.10.6.1 *Confinement and anti-buckling reinforcement*

The total effective area in each of the principal directions of the cross section within spacing s_h shall be greater than that given by Equation 10–22 or 10–23:

For confinement of concrete:

$$A_{sh} = \frac{(1 - p_t m)s_h h''}{3.3} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N^*}{\phi f_c' A_g} - 0.0065 s_h h'' \quad \text{(Eq. 10–22)}$$

where A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions, and $p_t m$ shall not be taken greater than 0.4.

where N^* is the maximum design axial load for load for load combinations involving wind or seismic actions, or any other load case in which significant lateral force is applied to the structure as a whole.

For lateral restraint of longitudinal bars against premature buckling:

No individual leg of a stirrup-tie shall be less than that given by Equation 10–23.

$$A_{te} = \frac{\Sigma A_b f_y}{135 f_{yt}} \frac{s_h}{d_b} \quad \text{(Eq. 10–23)}$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars between longitudinal bars restrained in accordance with 10.3.8.3.

In Equations 10–22 and 10–23, f_{yt} shall not be taken greater than 800 MPa.

10.3.10.6.2 *Spacing of tie sets*

The centre-to-centre spacing of the tie sets along the member shall be less than or equal to the smaller of one-third of the least lateral dimension of the cross section, or 10 diameters of the longitudinal bar being restrained.

10.3.10.6.3 *Support of longitudinal bars*

Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

(a) Bars or bundles of bars which lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed the larger of one-third of the lateral dimension of the cross section in the direction of the spacing or 200 mm;

(b) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside of hoop bars.

10.3.10.7 *Minimum diameter of transverse reinforcement*

10.3.10.7.1 *Minimum diameters for rectangular hoops and ties*

Rectangular hoop or tie reinforcement shall be at least 5 mm in diameter for longitudinal bars less than 20 mm in diameter, 10 mm in diameter for longitudinal bars from 20 to 32 mm in diameter and 12 mm in diameter for longitudinal bars larger than 32 mm in diameter and for bundled longitudinal bars.

10.3.10.7.2 *Minimum diameters for spiral and circular hoops*

Spiral or circular hoop reinforcement shall be of such a size and assembled so as to permit handling and placing without distortion from its designed dimensions. Spiral or circular hoop bar shall be equal to or greater than 5 mm in diameter.

10.3.10.8 *Anchorage of transverse reinforcement*

(a) Except where permitted by 8.7.2.8, transverse reinforcement shall not be anchored by lap splicing; A2

(b) Spirals shall be anchored by either welding to the previous turn, in accordance with 8.7.4.1(b) or by terminating the spiral with at least a 135° stirrup hook, engaging a longitudinal bar and with the stirrup hook being a clear distance away from the previous turn of not more than 25 mm;

(c) Circular or rectangular hoops shall be anchored by either a mechanical connection or welded splice in accordance with 8.7.4.1(b), or by terminating each end of the hoop with at least a 135° stirrup hook, overlapping the other end and engaging a longitudinal bar. Each end of a cross tie shall engage a longitudinal bar with at least a 135° stirrup hook.

10.3.10.9 *Set out of transverse reinforcement at column ends*

At the ends of columns and piers, the spacing of transverse reinforcement shall be:

(a) Located vertically not more than 75 mm above the top of the footing or slab in any storey, and not more than 75 mm below the lowest horizontal reinforcement in members supported above;

(b) Where beams or brackets do not frame into all sides of a column, ties shall extend above termination of spirals or circular hoops to bottom of slab or drop panel;

(c) In columns with capitals, transverse reinforcement shall extend to a level at which the diameter or width of capital is two times that of the column;

(d) For column bars that are not restrained against buckling by beams, the distance between the first tie in the column and that within the beam column joint shall not exceed six times the diameter of the column bar to be restrained.

10.3.11 ***Composite compression members***

10.3.11.1 *General*

Composite compression members shall include all such members reinforced longitudinally with structural steel shapes, pipes, or tubing with or without longitudinal bars.

10.3.11.2 *Strength*

Strength of a composite member shall be computed for the same limiting conditions applicable to ordinary reinforced concrete members.

10.3.11.3 *Axial load strength assigned to concrete*

Any axial load strength assigned to concrete of a composite member shall be transferred to the concrete by members or brackets in direct bearing on the composite member concrete.

10.3.11.4 *Axial load strength not assigned to concrete*

All axial load strength not assigned to concrete of a composite member shall be developed by direct connection to the structural steel shape, pipe, or tube.

10.3.11.5 *Slenderness effects*

Slenderness effects shall be provided for by methods based on a fundamental analysis. Alternatively they may be based on a radius or gyration of a composite section given by:

$$r = \sqrt{\frac{(E_c I_g / 5) + E_s I_t}{(E_c A_g / 5) + E_s A_t}} \qquad \text{(Eq. 10–24)}$$

and, as an alternative to a more accurate calculation, EI in Equation 10–5 shall be taken either as Equation 10–6 or:

$$EI = \frac{(E_c I_g / 5)}{1 + \beta_d} + E_s I_t \quad \text{(Eq. 10–25)}$$

10.3.11.6 *Structural steel encased concrete core*

10.3.11.6.1 *Steel encased concrete core*

For a composite member with a concrete core encased by structural steel, the thickness of the steel encasement shall be equal to or greater than:

$t_s > b\sqrt{\frac{f_y}{3E_s}}$ for each face of width b

or

$t_s > h\sqrt{\frac{f_y}{8E_s}}$ for circular sections of diameter h

where, for the purpose of this clause, f_y is the yield stress of the structural steel casing.

10.3.11.6.2 *Longitudinal bars*

Longitudinal bars located within the encased concrete core may be used in computing A_t and I_t.

10.4 Additional design requirements for members designed for ductility in earthquakes

10.4.1 *Strength calculations at the ultimate limit state*

The design of cross sections subjected to flexure with or without axial load shall be consistent with 7.4.2.

10.4.2 *Protection of columns at the ultimate limit state*

For frames where sidesway mechanisms with plastic hinges forming only in columns are not permitted at the ultimate limit state, the design moments and axial loads on columns shall include the effect of possible beam overstrength, concurrent seismic forces, and magnification of column moments due to dynamic effects, in order to provide a high degree of protection against the formation of a column sway mechanism.

10.4.3 *Dimensions of columns and piers*

10.4.3.1 *General*

For columns or piers, which sustain plastic regions in the ultimate limit state in load combinations involving seismic actions, either an analysis based on first principles shall be made to demonstrate that the member is stable, or the dimension limits given in 10.4.3.2, 10.4.3.3 and 10.4.3.4 shall be satisfied.

10.4.3.2 *Columns in framed structures*

The depth, width and clear length between the faces of supports of members with rectangular cross sections, to which moments are applied at both ends by adjacent beams, columns or both, shall be such that:

$$\frac{L_n}{b_w} \le 25 \quad \text{(Eq. 10–26)}$$

and

$$\frac{L_n h}{b_w^2} \le 100 \quad \text{(Eq. 10–27)}$$

10.4.3.3 *Cantilevered columns*

The depth, width and clear length from the face of support of cantilever members with rectangular cross sections, excluding bridge piers, shall be such that:

$$\frac{L_n}{b_w} \le 15 \quad \text{(Eq. 10–28)}$$

and

$$\frac{L_n h}{b_w^2} \le 60 \quad \text{(Eq. 10–29)}$$

10.4.3.4 *Web width of T - and L - member*

The width of web of T- and L- members, in which the flange or flanges are integrally built with the web, shall be such that the values given by Equations 10–26 and 10–28 are not exceeded by more than 50 %.

10.4.3.5 *Compression face width of T-, L- or - members*

The width of the compression face of a member with rectangular, T-, L- or I- section shall be greater than or equal to 200 mm.

10.4.3.6 *Narrow beams and wide columns*

Where narrow beams frame into columns, the width of a column that shall be assumed to resist the forces transmitted by the beam shall be in accordance with 15.3.4.

10.4.4 ***Limit for design axial force on columns and piers***

For columns and piers the maximum design load in compression, N_o^*, shall be less than $0.7N_{n,max}$ where:

$$N_{n,max} = \alpha_1 f_c' (A_g - A_{st}) + f_y A_{st} \quad \text{(Eq. 10–30)}$$

where α_1 is given by 7.4.2.7(c).

10.4.5 ***Ductile detailing length***

Ductile detailing lengths, ℓ_y, of end regions in columns and piers adjacent to moment resisting connections shall be the greater of:

(a) (i) $\ell_y = h$ for $N_o^* \le 0.25\,\phi f_c' A_g$,

(ii) $\ell_y = 2.0h$ for $0.25\phi f_c' A_g < N_o^* \le 0.5\phi f_c' A_g$

(iii) $\ell_y = 3.0h$ for $0.5\phi f_c' A_g < N_o^* \le 0.7\phi N_{n,max}$

where

h is the diameter of a circular cross section or the dimension in the direction resisting the applied moment of a rectangular section

or

(b) The length from the joint over which the design moment, taking into account dynamic magnification and overstrength actions, is greater than the following proportions of the moment at the end of the member:

(i) 0.8 for $N_o^* \le 0.25\,\phi f_c' A_g$

(ii) 0.7 for $0.25\phi f_c' A_g < N_o^* \le 0.5\phi f_c' A_g$

(iii) 0.6 for $0.5\phi f_c' A_g < N_o^* \le 0.7\phi N_{n,max}$

10.4.6 ***Longitudinal reinforcement in columns and piers***

10.4.6.1 *Longitudinal reinforcement*

Longitudinal reinforcement in columns and piers shall be as required by 10.4.6.2 to 10.4.6.7.

10.4.6.2 *Maximum area of longitudinal reinforcement*

The area of longitudinal reinforcement shall be not greater than $18A_g/f_y$ except that in the region of lap splices the total area shall not exceed $24A_g/f_y$.

10.4.6.3 *Spacing of longitudinal bars in plastic hinge region*

The spacing of longitudinal bars in potential plastic regions, as defined in 10.4.5, shall satisfy the appropriate limits given in (a) and (b) below.

(a) For a member with a circular cross section, the centre-to-centre spacing between longitudinal bars shall be less than or equal to the larger of one-quarter of the diameter of the section or 200 mm.

(b) In rectangular sections the maximum permissible centre-to-centre spacing of longitudinal bars, which are cross linked across the cross section, shall depend on the ratio of the longer side, h, to the shorter side, b, as set out in (i) and (ii);

  (i) Where the ratio of h/b < 2.0 the maximum permissible spacing shall be the larger of $b/4$ or 200mm;

  (ii) Where the ratio of $h/b \geq 2.0$ the maximum spacing shall be as for (i) except in the mid regions of the longer side. In the region lying between lines drawn at a distance of the larger of b or 1.5 times the depth to the neutral axis from the extreme fibres, the spacing may be increased to the smaller of $h/4$ or 300 mm. This region is defined by lines drawn at a distance, which is the greater of b or 1.5 times the depth to the neutral axis, from the extreme fibres of the section.

10.4.6.4 *Spacing of longitudinal reinforcement in columns*

The spacing of longitudinal bars given in 10.4.6.3 may be relaxed to those in 10.3.8.3 for the regions of the column defined below:

(a) for regions of column located between the ductile detailing length, ℓ_y, identified in 10.4.5.

(b) for columns designed using method A of Appendix D within the ductile detailing lengths defined in D3.1 (Method A) as providing a high level of protection against the formation of plastic regions.

10.4.6.5 *Anchorage of column bars in beam column joints*

10.4.6.5.1 *Termination of bars in potential plastic hinge regions*

Where column bars terminate in beam column joints or joints between columns and foundation members and where a plastic hinge in the column may be expected, the anchorage of the longitudinal column bars into the joint region shall be assumed to commence at one-half of the depth of the beam or $8d_b$, whichever is less, from the face at which the column bar enters the beam or foundation member. When it is shown that a column plastic hinge adjacent to the beam face cannot occur, the development length shall be considered to commence from the beam face of entry.

10.4.6.5.2 *Termination of bars in joint*

Notwithstanding the adequacy of the anchorage of a column bar into an intersecting beam, no column bar shall be terminated in a joint area without a horizontal 90° standard hook or equivalent anchorage device as near the far face of the beam as practically possible, and not closer than three-quarters of the depth of the beam to the face of entry. Unless a column is designed to resist only axial forces, the direction of the horizontal leg of the bend must always be towards the far face of the column.

10.4.6.6 *Maximum longitudinal column bar diameter in beam column joint zones*

The maximum diameter of longitudinal bars passing through a beam column joint zone shall satisfy the appropriate requirement of (a) or (b) given below:

(a) Where columns have been designed by Method B in Appendix D, or by Method A in Appendix D and the joint zone being considered is below the mid-height of the second storey:

$$\frac{d_b}{h_b} \leq 3.2\frac{\sqrt{f_c'}}{f_y} \qquad \text{(Eq. 10–31)}$$

(b) Where columns have been designed by Method A and the joint zone being considered is above the mid-height of the second storey, the maximum diameter is given by:

$$\frac{d_b}{h_b} \le 4.0\frac{\sqrt{f_c'}}{f_y} \quad \text{.....(Eq. 10–32)}$$

This requirement need not be met if it is shown that stresses in extreme column bars during an earthquake remain in tension or compression over the whole bar length contained within the joint.

10.4.6.7 *Detailing of column bars passing through beam column joints*

Longitudinal column bars passing through the joint must be extended straight through joints of the type covered by 10.4.6.6(a). Where longitudinal column bars within or near joints of the type covered by 10.4.6.6(b) are offset, the slope of the inclined bars with the axis of the column shall not exceed 1 in 6, and horizontal ties at the bend, in addition to those otherwise required by 15.4.4, shall be provided to carry 1.5 times the horizontal thrust developed by the column bars at yield stress.

10.4.6.8 *Splices of longitudinal reinforcement*

10.4.6.8.1 *General*

Splices in the longitudinal reinforcement of columns and piers shall comply with 8.7.

10.4.6.8.2 *Location of splices in reinforcement*

Full strength welded splices meeting the requirements of 8.7.4.1(a) may be used in any location. For all other splices the following restrictions apply:

(a) In a column in a building the centre of the splice must be within the middle quarter of the storey height of the column unless it can be shown that there is a high degree of protection against the formation of hinges adjacent to the beam faces;

(b) In a bridge column or pier no part of a splice shall be located within a distance of the member depth from a section where the reinforcement may reach $0.9f_y$ when overstrength moments act in the column;

(c) Reinforcement in columns of buildings or piers of bridges shall not be spliced by lapping in a region where stresses at the ultimate limit state may exceed $0.6f_y$ in tension or compression unless each spliced bar is confined by stirrup-ties so that:

$$\frac{A_{tr}}{s} \ge \frac{d_b f_y}{48 f_{yt}} \quad \text{.....(Eq. 10–33)}$$

10.4.7 ***Transverse reinforcement in columns and piers***

10.4.7.1 *Transverse reinforcement quantity*

Transverse reinforcement shall satisfy the requirements of shear, confinement of concrete and lateral restraint of longitudinal bars against premature buckling. The maximum area required for shear combined with torsion, confinement, or control of buckling of bars shall be used.

10.4.7.2 *Design for shear*

10.4.7.2.1 *Design shear force*

The design shear force of columns and piers subjected to combined flexure and axial load shall be determined from the consideration of forces on the member, with the combination of maximum likely end moments which gives the maximum shear.

The minimum nominal shear strength permitted in a column, at the ultimate state, shall be equal to or greater than:

(a) For a building with more than one storey, 1.6 times the shear force V_E;

(b) For a building with one storey or a bridge, 1.5 times V_E

(c) In the first storey of a building with two or more storeys, or in any structure where lateral seismic forces or elongation can cause plastic hinges to form at both ends of the member, the design shear shall not be less than the sum of the overstrength moments at each end divided by the clear distance between the critical sections of the plastic hinges.

(d) As required by the method A or B (in Appendix D) being used in the design of the columns.

A2 **10.4.7.2.2** *Types of potential plastic hinges in columns*

Detailing in the ductile detailing length shall be suitable for the type of potential plastic region defined in (a) or (b) below as appropriate:

(a) Columns shall be designed with ductile potential plastic regions where any of the following applies:
  (i) The columns have been designed with a low level of protection against the formation of plastic hinges as in Method B in Appendix D;
  (ii) Where the section ductility exceeds the limit for limited ductile plastic regions;
  (iii) In the lower one and a half storeys of multi-storey buildings designed with a high level of protection against the formation of plastic hinges as in Method A in Appendix D.

(b) Columns shall be designed with ductile or limited ductile plastic regions where either:
  (i) The section ductility is equal to or less than the limit for limited ductile plastic regions, or
  (ii) In columns in multi-storey buildings designed with a high level of protection against the formation of plastic hinges in levels above the first one and a half storeys as in Method A in Appendix D.

10.4.7.2.3 *Design of shear reinforcement*

The design of shear reinforcement in columns shall comply with 7.5 and 10.3.10.2, 10.3.10.3 and 10.3.10.4, except where specifically noted in (a) or (b) as appropriate below:

(a) In regions outside ductile detailing lengths, 10.3.10.4.4 shall be replaced by 10.4.7.2.7;

(b) Within the ductile detailing lengths of columns the design of shear reinforcement shall be either by:
  (i) The strut and tie method with 10.3.10.2.2(a) being replaced by 10.4.7.2.4 and 10.3.10.4.4 replaced by 10.4.7.2.7; or
  (ii) Satisfying the requirements of 10.4.7.2.5.

10.4.7.2.4 *Strut and tie method for shear design*

A2 Where the strut and tie method is used within ductile detailing lengths, the following conditions shall apply:

(a) The shear resistance of the concrete in the flexural tension zone shall be taken as zero;

(b) The angle between the diagonal compression struts and the flexural tension reinforcement shall be equal to or greater than 45°.

10.4.7.2.5 *Conventional method of shear design*

Shear resistance provided by concrete shall be computed from 10.3.10.3, but with 10.3.10.3.1 being replaced by 10.4.7.2.6.

Shear resistance provided by reinforcement shall satisfy the requirements of 10.3.10.4, but 10.3.10.4.4 shall be replaced by 10.4.7.2.7.

10.4.7.2.6 *Nominal shear stress provided by the concrete in columns or piers*

The nominal shear strength provided by concrete in columns shall be taken as the product of the shear area, A_{cv}, times the nominal shear stress, v_c, given in (a), (b) or (c) as appropriate.

(a) In regions outside ductile detailing lengths, the nominal shear stress resistance of concrete, v_c, is given by 10.3.10.3.1;

(b) Within the ductile detailing length for ductile plastic regions, the nominal shear stress resistance of concrete, v_c is given by:

$$v_c = 3v_b\sqrt{\left[\frac{N_o^*}{A_g f_c'} - 0.1\right]} \geq 0.0 \qquad \text{(Eq.10–34)}$$

(c) Within the ductile detailing length for limited ductile plastic regions, the nominal shear stress resistance of concrete, v_c, is given by:

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$$v_c = v_b\left[0.5+3\left(\frac{N_o^*}{A_g f_c'}-0.1\right)\right] \geq 0.0 \quad \text{.....(Eq.10–35)}$$

$$v_c = 0.0 \text{ for } \frac{N_o^*}{A_g f_c'} \leq -0.067 \quad \text{.....(Eq.10–36)}$$

where v_b is given by 10.3.10.3.1 and N_o^* is taken as positive for compressive axial loads.

10.4.7.2.7 *Minimum shear reinforcement*

The area of shear reinforcement shall be equal to or greater than:

$$A_v = \frac{1}{12}\sqrt{f_c'}\,\frac{b_w s}{f_{yt}} \quad \text{.....(Eq. 10–37)}$$

10.4.7.3 *Alternative design methods for concrete confinement and lateral restraint of longitudinal bars*

In lieu of the methods specified in 10.4.7.4 and 10.4.7.5, moment-curvature analysis may be conducted in order to achieve the calculated curvature ductility factor in the potential plastic hinge regions at the ultimate limit state. Such an analysis shall be conducted using stress-strain relations for confined concrete and reinforcing steel, satisfying the requirements of equilibrium and compatibility of strains with allowance for reversed cyclic loading, to determine the transverse reinforcement required for concrete confinement and lateral restraint of longitudinal bars against premature buckling.

10.4.7.4 *Design of spiral or circular hoop reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.4.7.4.1 *In ductile potential plastic hinge regions*

In the ductile detailing lengths of potential plastic regions as defined in 10.4.5 and 10.4.7.2.2, where spirals or circular hoops are used, the volumetric ratio, p_s, shall be equal to or larger than that given by the greater of Equation 10–38 or Equation 10–39. A2

(a) For confinement of concrete A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions. The required confinement reinforcement is given by:

$$p_s = \frac{(1.3-p_t m)}{2.4}\frac{A_g}{A_c}\frac{f_c'}{f_{yt}}\frac{N_o^*}{f_c' A_g} - 0.0084 \quad \text{.....(Eq. 10–38)}$$

Where the value of $p_t m$ used in the equation shall not be taken greater than 0.4.

(b) For lateral restraint of longitudinal bars against premature buckling:

$$p_s = \frac{A_{st}}{110 d''}\frac{f_y}{f_{yt}}\frac{1}{d_b} \quad \text{.....(Eq. 10–39)}$$

In Equations 10–38 and 10–39 f_{yt} shall not exceed 800 MPa.

10.4.7.4.2 *In limited ductile plastic hinge regions*

Where spiral or circular hoops are used in the ductile detailing lengths defined in 10.4.5, for limited ductile plastic regions as defined in 10.4.7.2.2(b), the quantity of transverse reinforcement provided shall be equal to or larger than the greater of that given by Equation 10–39, or 70 % of that required by Equation 10–38. A2

10.4.7.4.3 *In columns protected against plastic hinging*

A2 In frames where columns are designed with sufficient strength to provide a high degree of protection against plastic hinging, the required quantity of transverse reinforcement placed in the regions of columns defined as ductile detailing lengths in potential plastic hinge regions in 10.4.5 shall be the larger of that required by Equation 10–21, or 70 % of that required by Equation 10–38. This reduction in the quantity of transverse reinforcement in potential plastic hinge regions shall not be permitted at the top and bottom of the columns below the mid height of the first elevated storey (one and a half storeys above the primary plastic hinges at the base of the columns), nor in a column in any other storey where the column may form a primary plastic hinge at the top and/or bottom of the column in the storey.

10.4.7.4.4 *In regions outside potential plastic hinge regions*

A2 Outside the plastic hinge ductile detailing lengths defined in 10.4.5, transverse reinforcement shall be as required by 10.3.10.5.

10.4.7.4.5 *Spacing of spirals or circular hoop reinforcement in columns and piers*

When spiral or circular hoop reinforcement is used the spacing shall be as follows:

A2 (a) In a ductile detailing length of a ductile potential plastic hinge region as defined by 10.4.5, and 2.6.1.3 the centre-to-centre spacing of spirals or circular hoops along the member shall be equal to or less than the smaller of one-quarter of the diameter of the cross section of the member or six times the diameter of the longitudinal bar to be restrained;

A2 (b) In a ductile detailing length of a limited ductile potential plastic hinge regions and in potential plastic hinge regions with a high degree of protection against plastic hinging, the centre-to-centre spacing of spirals or circular hoops along the member shall be equal to or less than the smaller of one-quarter of the diameter of the cross section of the member or ten times the diameter of the longitudinal bar to be restrained;

A2 (c) Outside the ductile detailing lengths of potential plastic hinge regions and in potential plastic hinge regions with a high degree of protection against plastic hinging, the centre-to-centre spacing of transverse reinforcement along the member shall be equal to or less than the smaller of one-third of the diameter of the column or ten times the diameter of the longitudinal bar to be restrained.

10.4.7.5 *Design of rectangular hoop or tie reinforcement for confinement of concrete and lateral restraint of longitudinal bars*

10.4.7.5.1 *In ductile potential plastic hinge regions*

A2 In the ductile detailing lengths as defined in 10.4.5 of potential ductile plastic regions as defined in 10.4.7.2.2, where rectangular hoops or ties are used the total effective area of hoop bars and supplementary cross-ties in each of the principal directions of the cross section within spacing, s_h, shall be the greater of Equation 10–40 or Equation 10–41.

(a) For confinement of concrete A_g/A_c shall not be greater than 1.5 unless it can be shown that the design strength of the column core can resist the design actions. The required confinement reinforcement is given by:

$$A_{sh} = \frac{(1.3 - p_t m) s_h h''}{3.3} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N_o^*}{\phi f_c' A_g} - 0.006 s_h h'' \quad \text{(Eq. 10–40)}$$

Where the value of $p_t m$ used in the equation shall not be taken greater than 0.4.

(b) For lateral restraint of longitudinal bars against premature buckling:

The area of each leg of a hoop bar or cross tie in the direction of potential buckling of the longitudinal bar shall be given by:

$$A_{te} = \frac{\Sigma A_b f_y}{96 f_{yt}} \frac{s_h}{d_b} \quad \text{.....(Eq. 10–41)}$$

where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 10.4.7.6. Longitudinal bars centred more than 75 mm from the inner face of stirrup-ties need not be considered in determining the value of ΣA_b.

In Equations 10–40 and 10–41 f_{yt} shall not be taken greater than 800 MPa.

10.4.7.5.2 *In limited ductile plastic hinge regions*

In the ductile detailing length as defined in 10.4.5 of potential limited ductile plastic regions as defined in 10.4.7.2.2, the transverse reinforcement provided shall be the larger of that required by Equation 10–41 or 70 % of that required by Equation 10–40. A2

10.4.7.5.3 *In potential plastic hinge regions with protection against plastic hinging*

In frames where columns are designed with sufficient strength to provide a high degree of protection against plastic hinging (method A in Appendix D, above mid-height of second storey), the required quantity of transverse reinforcement placed in the regions of columns defined as potential plastic hinge regions in 10.4.5 shall be that larger of that required by Equation 10–23 or 70 % of that required by Equation 10–40. This reduction in the quantity of transverse reinforcement in potential plastic hinge regions shall not be permitted at the top and bottom of the columns of the first storey nor in any storey in which a column sidesway mechanism could occur with plastic hinges forming in the columns. A2

10.4.7.5.4 *In regions outside the potential plastic hinge regions*

Outside the ductile detailing lengths defined in 10.4.5, transverse reinforcement shall be as required by 10.3.10.6 as appropriate. A2

10.4.7.5.5 *Spacing of rectangular hoop or tie reinforcement in columns and piers*

When rectangular hoop or tie reinforcement is used the spacing shall be as follows:

(a) In ductile plastic regions defined in 10.4.7.2.2 in the associated ductile detailing lengths as defined in 10.4.5, the centre-to-centre spacing of stirrup-ties shall not exceed the smaller of one-quarter of the least lateral dimension of the cross section of the member or six times the diameter of any longitudinal bar to be restrained in the outer layers; A2

(b) In limited ductile plastic regions defined in 10.4.7.2.2 in the associated ductile detailing lengths as defined in 10.4.5, the centre-to-centre spacing of stirrup ties shall not exceed the smaller of one-quarter of the least lateral dimension of the members or ten times the diameter of any longitudinal bar to be restrained in the outer layers; A2

(c) Outside the ductile detailing lengths of potential plastic hinge regions of a column or pier and in potential plastic hinge regions with a high degree of protection against plastic hinging, over the length of the column or pier between the potential plastic hinge regions, the centre-to-centre spacing of transverse reinforcement along the member shall not exceed the smaller of one-third of the least lateral dimension, or ten times the diameter of the longitudinal bar to be restrained. A2

10.4.7.6 *Support of longitudinal bars*

In potential plastic hinge regions, each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

(a) Bars or bundles of bars which lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed the larger of one-quarter of the adjacent lateral dimension of the cross section or 200 mm between centres;

(b) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside of hoop bars.

NOTES

11 DESIGN OF STRUCTURAL WALLS FOR STRENGTH, SERVICEABILITY AND DUCTILITY

11.1 Notation

A_c^* area of concrete core extending over the outer c' length of the neutral axis depth which is subjected to compression, measured to centre of peripheral hoop legs, mm^2

A_{cv} area used to calculate shear area, taken as td, where t is the minimum wall thickness, mm^2 A2

A_g gross area of section, mm^2

A_g^* gross area of concrete section extending over outer c' length of the neutral axis depth which is subjected to compression, mm^2

A_r h_w/L_w aspect ratio of wall

A_s area of longitudinal (vertical) reinforcement at a spacing of s_v along the wall, mm^2 A2

A_{sh} total effective area of hoop bars and supplementary cross ties distributed over length h'' in the direction under consideration, within vertical spacing s_h, mm^2

A_t total area of longitudinal reinforcement at a section in a wall, mm^2 A2

A_{te} area of one leg of stirrup-tie, mm^2

A_v horizontal shear reinforcement. mm^2

A_{wb} gross area of boundary element, mm^2

b width or thickness of wall section, mm

b_m thickness of boundary region of wall at potential plastic hinge region, mm

b_w web width, mm

c computed distance of neutral axis from the extreme compression fibre of the wall section at the nominal flexural strength limit state, mm

c' length of wall section defined by Equation 11–27 to be confined by transverse reinforcement, mm

c_b distance from extreme compression fibre to neutral axis at balanced strain conditions

c_c a limiting depth given by Equation 11–25, mm A2

d distance from extreme compression fibre to centroid of tension force in longitudinal reinforcement, which may be taken as defined in 11.3.10.3.3 for shear strength calculations, mm

d_b diameter of the longitudinal bar to be restrained, mm

$d_{b\ell}$ diameter of the longitudinal reinforcement, mm

E_c modulus of elasticity of concrete, MPa A2

f'_c specified compressive strength of concrete, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yh} lower characteristic yield strength of non-prestressed hoop or supplementary cross tie reinforcement, MPa

f_{yn} lower characteristic strength of vertical non-prestressed reinforcement, MPa

f_{yt} transverse reinforcement yield strength, MPa

h'' dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop, mm

h_w total height of wall from base to top, mm

I moment of inertia at section, mm^4

I_{cr} second moment of area of transformed cracked section, mm^4 A2

k_e effective length factor for Euler buckling

k proportion of the neutral axis depth to the effective depth of member in elastically responding transformed section, calculated neglecting the axial load A2

k_{ft} effective length factor for flexural torsional buckling

k_m factor for determining b_m

L_b length of flexural member, mm

L_c length of compression member, mm

L_d development length, mm

L_n the clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span, mm

L_p the length of the plastic hinge, mm

L_w horizontal length of wall, mm

M° overstrength moment of resistance of the section at the base of a cantilever wall, N mm

M^* design moment, N mm

A2 M_e^* design moment at the base of the wall corresponding with μ = 1.25 and S_p = 0.9, N mm

N^* design axial load at the ultimate limit state, N

n modular ratio E_s/E where E_s is the modulus of elasticity of steel

A2

p_ℓ The ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section = A_s/ts_v

p_n Ratio of vertical (longitudinal) wall reinforcement area to gross concrete area of horizontal section, A_t/A_g

s centre-to-centre spacing of shear reinforcement along member, mm

s_1 centre-to-centre spacing of vertical shear reinforcement, mm

s_2 centre-to-centre spacing of horizontal shear reinforcement, mm

s_h centre-to-centre spacing of horizontal hoop sets, mm

s_v horizontal spacing of vertical reinforcement along the length of a wall, mm

t wall thickness, mm

V^* design shear force, N

v_c shear resisted by concrete, MPa

V_c concrete shear strength, N

V_n total nominal shear strength, N

V_s nominal shear strength provided by shear reinforcement, N

α_m factor for determining wall slenderness

α_t factor for determining thickness of boundary section of wall

β factor for determining ductility factor

ε_c extreme fibre compression strain

ϕ_{ow} ratio of moment of resistance at overstrength to moment resulting from specified earthquake actions, where both moments refer to the base section of wall

λ factor for determining wall slenderness

μ displacement ductility capacity relied on in the design of the wall element

ΣA_b sum of area of longitudinal bars, mm^2

ξ factor for determining thickness of boundary section of wall

ψ ratio of $\Sigma EI/L_c$ of compression members to $\Sigma EI/L_b$ of flexural members in a plane at one end of a compression member

ψ_{min} the smaller of ψ_A or ψ_B which represents the ψ ratio at each end, A and B, of a compression member

11.2 Scope

11.2.1 *Application*

Provisions of this section shall apply to the design of walls subjected to axial load, with or without flexure, and shear. The provisions of this and earlier sections are summarised in Table C11.1. The clauses within Section 11 take precedence over Table C11.1.

11.2.2 *Requirements determined by curvature ductility*

A2 Walls containing plastic regions with sectional curvature ductility demands at the ultimate limit state less than or equal to the limits for nominally ductile plastic regions defined in 2.6.1.3 shall meet the requirements of 11.3 whilst walls in structures designed for greater curvature ductility than this shall be designed to meet the requirements of 11.3 as modified by 11.4.

11.3 General principles and design requirements for structural walls

11.3.1 *General design principles*

11.3.1.1 *General*

Walls shall be designed for any vertical loading and/or lateral in-plane and face forces to which they may be subjected. The design moment, M^*, for bending about the weak axis of the wall, shall include consideration of the additional moment caused by the eccentricity of the applied axial load to the expected deflected shape.

11.3.1.2 *Provision for eccentric loads*

The design of a wall shall take account of the actual eccentricity of the vertical force but in no case shall the design bending moment (M^*) be taken as less than N^* times $0.05t$.

11.3.1.3 *Effective flange projections for walls with returns*

Where wide flanges are present in relatively short walls, only the vertical reinforcement placed within a flange width, each side of the web, equal to one-half the distance from the section under consideration to the top of the wall shall be considered effective in resisting flexure.

11.3.2 *Minimum wall thickness*

Structural walls shall have a thickness, t, equal to or greater than 100 mm.

11.3.3 *Maximum wall thickness for singly reinforced walls*

Basement walls more than 250 mm thick and other walls more than 200 mm thick shall have the reinforcement placed in two layers parallel with the faces of the wall.

11.3.4 *Design for stability*

11.3.4.1 *Design by rational analysis*

Walls shall be designed to ensure stability at the ultimate limit state due to:

(a) P-delta effects associated with bending about the weak axis of the wall
(b) Euler buckling
(c) Flexural torsional buckling

Stability may be determined by rational analysis using the assumption in 11.3.4.2, or by simplified methods outlined in 11.3.4.3.

11.3.4.2 *Design assumptions for rational stability analysis*

The design moment determined by rational analysis shall consider the eccentricity of the applied load, degree of support fixity, and a wall stiffness calculated from transformed sectional properties neglecting concrete in tension, multiplied by a stiffness reduction factor of 0.75 (shown in brackets in Equations 11–3 and 11–4). A2

11.3.4.3 *Simplified methods of stability analysis*

11.3.5 provides simplified methods of ensuring stability at the ultimate limit state for slender walls with a single layer of centrally placed reinforcement.

11.3.6 provides a simplified method of ensuring stability in doubly reinforced walls subjected to eccentric axial load without face loads.

11.3.5 *Simplified stability assessment for slender singly reinforced walls*

11.3.5.1 *Design for actions causing bending about the weak axis*

11.3.5.1.1 *Limitations on use of method*

Walls designed using the requirements of 11.3.5.1.2 shall:

(a) Have a vertical stress N^*/A_g at the mid-height of the section of less than $0.06 f'_c$, for the load case causing bending about the weak axis.
(b) The walls shall be supported at the top and bottom. The method is not applicable to cantilevered walls bent about their weak axis.

11.3.5.1.2 *Design moment and P-delta effects – simplified method*

The design moment strength ϕM_n for combined flexure and axial loads at the mid-height cross section shall satisfy:

$\phi M_n \geq M^*$ .....(Eq. 11–1)

where

$M^* = M_a^* + N^* \Delta_u$ .....(Eq.11–2)

M_a^* is the moment at the mid-height section of the wall due to factored loads, and Δ_u is:

$$\Delta_u = \frac{5M^* L_n^2}{(0.75)48E_c I_{cr}}$$ .....(Eq. 11–3)

M^* shall be obtained by iteration of deflections, or by direct calculation using Equation 11–4

$$M^* = \frac{M_a^*}{1 - \frac{5N^* L_n^2}{(0.75)48E_c I_{cr}}}$$ .....(Eq. 11–4)

Where

For short-term loads E_c is given by 5.2.3, and for long-term loads E_c shall be modified to consider creep.

A2 The value of I_{cr} shall either be calculated by rational analysis based on elastic analysis, or the approximation given by Equation 11–5 may be used where $N^* / A_g f_c' \leq 0.06$.

$$I_{cr} = nA_{se}(d - kd)^2 + \frac{L_w(kd)^3}{3}$$ .....(Eq. 11–5)

where k is determined by elastic theory,

$$n = \text{modular ratio} = \frac{E_s}{E_c}$$ .....(Eq. 11–6)

and A_{se} may be taken as:

$$A_{se} = \frac{N^* + A_t f_y}{f_y}$$ .....(Eq. 11–7)

11.3.5.2 *Design for actions causing bending about the strong axis*

11.3.5.2.1 *Limitation on use of method*

A2 Walls shall be designed to the requirements of 11.3.5.2.2 and shall:

(a) Have an axial load at the base of the wall of $N^* < 0.015 f_c' A_g$ for the load case causing bending about the strong axis.

(b) The eccentricity of the axial load from the longitudinal axis shall be less than the wall thickness.

(c) Singly reinforced walls that are designed to be part of the primary lateral load resisting system for in-plane loads, shall be designed to ensure that mid-height hinges shall not form in the walls due to face loading. Singly reinforced walls designed to resist face loads by cantilever action shall be designed to ensure that plastic hinges do not form at the base of the wall due to face loads.

11.3.5.2.2 *Prevention of flexural torsional buckling of walls loaded in-plane with low axial loads*

The limiting effective height to thickness ratio to prevent flexural torsional buckling shall be determined from the lesser of Equations 11–8, 11–9 and 11–10.

$$\frac{k_{ft}L_n}{t} \le 12\sqrt{\frac{L_n / L_w}{\lambda}} \quad \text{(Eq. 11–8)}$$

and

$$\frac{L_n}{t} \le 75 \quad \text{(Eq. 11–9)}$$

and

$$\frac{k_{ft}L_n}{t} \le 65 \quad \text{(Eq. 11–10)}$$

where the effective length factor for flexural torsional buckling, k_{ft} is given by 11.3.5.2.3, and λ = the lesser of:

(a) $\dfrac{N^*}{f_c' A_g} + p_n \dfrac{f_y}{f_c'}$ A2

(b) $\dfrac{2.2M_e^*}{L_w A_g f_c'}$

where M_e^* is the design moment at the base of the wall due to lateral loads, for seismic load cases this shall be the action corresponding with $\mu = 1.25$ and $S_p = 0.9$. A2

11.3.5.2.3 *Effective height between lines of lateral support*

The effective height between lines of lateral support for flexural torsional bucking shall be taken as $k_{ft}L_n$, where k_{ft} is given by Table 11.1.

Table 11.1 – Effective wall height co-efficient k_{ft}

Case	Support condition at		Potential plastic region classification for in-plane loads(4)	k_{ft}
	Base of wall	Top of wall		
1	Fixed	Pinned	NDPR	0.85 or 1.0 where out of plane hinge forms at base
2	Fixed	Pinned	LDPR, DPR	1.0
3	Fixed	Nil	NDPR	1.4
4	Fixed	Nil	LDPR, DPR	k_{ft} = 1.4 and $\frac{kL_n}{b} \le 30$
5	Pinned	Pinned	NDPR, LDPR, DPR	1.0

NOTE –
(1) Fixed, means rotational, lateral, and torsional support are provided.
(2) Pinned, means torsional and lateral restraint is provided, but not rotational restraint.
(3) Nil, means none of torsional, rotational, or lateral restraint is provided
(4) Abbreviations for potential plastic region classifications (see 2.6.1.3):
Nominally ductile plastic region, NDPR;
Limited ductile plastic region, LDPR;
Ductile plastic region, DPR.

11.3.6 *Simplified stability assessment for doubly reinforced concrete walls*

11.3.6.1 *Limitation on the use of the method*

Wall designed using the requirements of 11.3.6.2 shall:

(a) Be doubly reinforced;

(b) Not be subjected to face loads for the action combination being considered. A2

11.3.6.2 *Design for Euler buckling from eccentric axial loads*

Design of walls for loads eccentric to the wall longitudinal axis without face loads shall include the effects of slenderness using the method outlined in 10.3.2 when the unsupported height, (L_n), to wall thickness, t,
A2 | ratio exceeds the following limits:

$$\frac{k_e L_n}{t} \geq \frac{\alpha_m}{\sqrt{\frac{N^*}{f_c' A_g}}} \qquad \text{(Eq. 11–11)}$$

where

A2 | (a) α_m = 6.5 for walls braced against sidesway and pinned at each end;
(b) α_m = 8 for braced walls rotationally fixed at one end.

The effective length factor for Euler buckling, k_e, is given by:

(a) $k_e = 0.85 + 0.05\psi_{min}$ for walls braced against lateral sidesway; and

(b) $k_e = 2.0 + 0.3\psi$ for cantilevered walls not prevented from sidesway;

where

ψ = ratio of $\Sigma EI/L_c$ of compression members to $\Sigma EI/L_b$ of flexural members in a plane at one end of a compression member

ψ_{min} = the smaller of ψ_A or ψ_B which represent the ψ ratio at each end, A and B, of a compression member.

11.3.7 *Walls with high axial loads*

A2 | The ratio of effective height to thickness ($k_e L_n/t$) shall be equal to or less than 30 where $N^* > 0.2\, f_c' A_g$.

11.3.8 *Flexural crack control*

Walls subject to flexure shall be designed to control cracking in accordance with 2.4.4.

11.3.9 *Strength of walls in flexure*

The design of walls for flexure at the ultimate limit state shall be based on the assumptions given in 7.4 and on the satisfaction of conditions of equilibrium and compatibility of strains.

11.3.10 *Strength of walls in shear*

11.3.10.1 *General*

The design of walls for shear at the ultimate limit state shall be in accordance with 7.5.

11.3.10.2 *Shear design of face loaded walls*

Design for shear forces perpendicular to face of a wall shall be in accordance with the provisions for slabs in 12.7.

11.3.10.3 *Design for shear in the plane of a wall*

Design for horizontal shear forces in the plane of a wall shall be in accordance with 11.3.10.3.1 to 11.3.10.3.8.

11.3.10.3.1 *Design horizontal section for shear*

A2 | Design of a horizontal section for shear in the plane of a wall shall be based on 7.5, where concrete shear strength, V_c, shall be in accordance with 11.3.10.3.4 or 11.3.10.3.5 and shear reinforcement shall be in accordance with 11.3.10.3.8.

11.3.10.3.2 *Maximum nominal shear stress*

Total nominal shear stress, v_{max}, at any horizontal section for shear in the plane of a wall and based on the minimum net wall thickness shall not be taken greater than the value given by 7.5.2.

11.3.10.3.3 *Definition of d*

For design for horizontal shear forces in the plane of a wall, d shall be taken as equal to $0.8L_w$. A larger value of d, equal to the distance from the extreme compression fibre to the centre of force of all reinforcement in tension, may be used when determined by a strain compatibility analysis prior to first yielding of longitudinal reinforcement. For face loading, d shall be taken as the distance from the extreme compression fibre to the centroid of longitudinal tension reinforcement. A2

11.3.10.3.4 *Concrete shear strength – simplified*

The shear resistance provided by the concrete may be calculated by the simplified method given below in lieu of the more detailed method in 11.3.10.3.5. This simplified method may only be used where the ratio, p_ℓ, of longitudinal reinforcement to area of concrete for any part of the wall exceeds a value of 0.003 and the spacing of reinforcement does not exceed 300 mm in any direction. Where this condition is satisfied v_c shall be taken to be the smaller of:

$$V_c = 0.17\sqrt{f_c'}\,A_{cv} \quad \text{(Eq. 11–12)}$$ A2

or

$$V_c = 0.17\left[\sqrt{f_c'} + \frac{N^*}{A_g}\right]A_{cv} \quad \text{(Eq. 11–13)}$$ A2

where N^* is taken as negative for axial tension.

11.3.10.3.5 *Concrete shear strength – detailed*

Concrete shear strength, v_cA_{cv}, shall be computed by Equation 11–15 where v_c shall be the lesser of that calculated from Equations 11–14 and 11–15:

$$v_c = \left(0.27\sqrt{f_c'} + \frac{N^*}{4A_g}\right) \quad \text{(Eq.11–14)}$$

or

$$v_c = 0.05\sqrt{f_c'} + \frac{L_w\left(0.1\sqrt{f_c'} + 0.2\dfrac{N^*}{A_g}\right)}{\dfrac{M^*}{V^*} - \dfrac{L_w}{2}} \quad \text{(Eq.11–15)}$$

where N^* is negative for tension. When $(M^*/V^* - L_w/2)$ is zero or negative, Equation 11–15 shall not apply.

11.3.10.3.6 *Shear design of sections near base of walls*

Sections located closer to the wall base than a distance $L_w/2$ or one-half the wall height, whichever is less, shall be designed for the same V_c as that computed at a distance $L_w/2$ or one-half the height.

11.3.10.3.7 *Shear reinforcement always to be provided*

Irrespective of whether the total nominal shear strength, V_n, is more or less than $V_c/2$, reinforcement shall be provided in accordance with 11.3.10.3.8.

11.3.10.3.8 *Design of shear reinforcement*

Design of shear reinforcement for walls shall satisfy the following requirements:

(a) Where the total design shear force, V^*, exceeds the concrete shear strength, V_c, horizontal shear reinforcement shall be computed from:

$$V_s = \frac{V^*}{\phi} - V_c \quad \text{(Eq. 11–16)}$$

where

$$V_c = v_c A_{cv} \quad \text{(Eq. 11–17)}$$

and

$$V_s = A_v f_{yt} \frac{d}{s_2} \quad \text{(Eq. 11–18)}$$

where A_v is the area of horizontal shear reinforcement within a distance s_2.

(b) Irrespective of the requirements of (a) above the area of horizontal shear reinforcement in a wall shall be equal to or greater than:

$$A_v = \frac{0.7 b_w s_2}{f_{yt}} \quad \text{(Eq. 11–19)}$$

(c) Spacing of horizontal shear reinforcement, s_2, shall not exceed $L_w/5$, $3t$, or 450 mm; and

(d) Ratio p_n of vertical reinforcement area to gross concrete area of horizontal section shall be equal to or greater than $0.7/f_{yn}$; and

(e) Spacing of vertical shear reinforcement s_1 shall not exceed $L_w/3$, $3t$, or 450 mm.

11.3.11 *Wall reinforcement*

11.3.11.1 *General*

All concrete walls shall have reinforcement placed in two directions at an angle of approximately 90°. Bars shall not be bent round re-entrant angles unless special provisions are made for positive resistance of bursting forces at bends of bars.

11.3.11.2 *Placement of reinforcement in walls*

(a) Basement walls more than 250 mm thick and other walls more than 200 mm thick shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall.

(b) Bars shall be equal to or larger than 10 mm in diameter.

A2 | (c) Bars shall be spaced at no more than three times the thickness of the wall or 450 mm on centres, whichever is the least.

(d) The diameter of the bar in the wall shall not exceed one seventh of the wall thickness.

11.3.11.3 *Minimum and maximum area of reinforcement*

A2 | The ratio of vertical reinforcement in any section of a wall shall satisfy the limitations given in (a), (b) and (c) below:

(a) For actions causing bending about the weak axis of singly reinforced walls, the area of vertical reinforcement shall be such that at every section the distance from the extreme compression fibre to the neutral axis shall be equal to or less than $0.75c_b$;

(b) The ratio of vertical reinforcement to concrete unit area, p_ℓ, in a rectangular wall, or in a boundary element (11.4.2.3), or any other rectangular element in a wall, shall be equal to or less than $16/f_y$, except in regions where lapped splices in boundary elements are unavoidable, in which case the total ratio including the area of splices shall not exceed $21/f_y$;

(c) The ratio of vertical reinforcement to gross cross section area, p_n, shall be equal to or greater than $\frac{\sqrt{f_c'}}{4f_y}$, unless the area of vertical reinforcement is at least one third greater than that required by analysis, for both positive and negative moments, but in no case shall it be less than the greater of $0.7\ A_g/f_y$ or $0.0014A_g$. A2

11.3.11.4 *Reinforcement around openings*

In addition to the minimum as prescribed in 11.3.11.3 there shall be reinforcement with a yield strength equal to or greater than 600 N per mm of wall thickness, around all window or door openings. Such bars shall extend at least 600 mm beyond the corners of the openings.

11.3.11.5 *Ties around vertical reinforcement*

Vertical wall compression reinforcement shall be enclosed by lateral ties when the vertical reinforcement area equals or exceeds 0.01 times the gross concrete area in any locality of the wall section.

11.3.11.6 *Curtailment of flexural reinforcement*

Curtailment of flexural reinforcement shall comply with 8.6.12.

11.4 Additional design requirements for members designed for ductility in earthquakes

11.4.1 *General seismic design requirements*

11.4.1.1 *Interaction of flanges, boundary members and webs*

Cantilever or coupled structural walls shall be considered as integral units. The strength of flanges, boundary members and webs shall be evaluated on the basis of compatible interaction between these elements using rational analysis. Due allowance for openings in components shall be made.

11.4.1.2 *Design of ductile walls*

In the design of ductile walls subjected to seismic forces at the ultimate limit state, the requirements of 2.6.8 shall be satisfied.

11.4.1.3 *Effective flange projections for walls with returns*

For determining the nominal moment strength, M_n, of a wall the provisions of 11.3.1.3 shall apply.

When the overstrength moment of resistance, M°, is required, the effective width of the flange acting in tension, either side of the web, shall be equal to the distance from the section under consideration to the top of the wall but not greater than flange width. When the flange is in compression the requirements of 11.3.1.3 apply. A2

11.4.2 *Dimensional limitations*

11.4.2.1 *Prevention of buckling of thin walls loaded in-plane*

To safeguard against premature out of plane buckling in the potential plastic hinge region of walls with thin sections more than two storeys high, the limitations of 11.4.2.2 to 11.4.2.4 shall apply.

11.4.2.2 *Minimum thickness for prevention of instability within plastic hinge region*

To safeguard against out of plane buckling in the potential plastic hinge regions of ductile walls, the following limitations shall apply for walls with axial force levels greater than $0.05\ f_c' A_g$ and for ductile or limited ductile plastic region.

The thickness in the boundary region of the wall section, extending over the lesser of the plastic hinge length or the full height of the first storey, shall be equal to or greater than:

$$b_m = \frac{\alpha_r k_m \beta (A_r + 2) L_w}{1700\sqrt{\xi}} \quad \text{(Eq. 11-20)}$$

where

α_r = 1.0 for doubly reinforced walls and 1.25 for singly reinforced walls; and
β = 5 for limited ductile plastic regions
β = 7 for ductile plastic regions
k_m = 1.0, unless it can be shown that for long walls:

$$k_m = \frac{L_n}{(0.25 + 0.055A_r)L_w} < 1.0 \quad \text{(Eq. 11–21)}$$

and

$$\xi = 0.3 - \frac{p_\ell f_y}{2.5f_c'} > 0.1 \quad \text{(Eq. 11–22)}$$

11.4.2.3 *Dimensions of enlarged boundary element*

Where 11.4.2.2 controls the thickness of the wall in the boundary region, an enlarged boundary element shall be provided with gross area, A_{wb}, satisfying the following limitations:

$$b_m^2 \le A_{wb} \ge \frac{b_m L_w}{10} \quad \text{(Eq. 11–23)}$$

11.4.2.4 *Flange thickness*

Where flanges on either side of the web with width greater than three times the flange thickness, satisfying
A2 | 11.4.2.2 are used, the effective length to flange thickness ratio ($k_e L_n/b$) shall not exceed 30.

11.4.3 *Ductile detailing lengths*

The ductile detailing length in potential plastic regions in walls subjected to in plane loading shall be taken as the length of the wall, L_w, or 0.17 M / V in the critical ultimate strength load combination which includes seismic actions, whichever is larger, measured from the section at which the first flexural yielding is expected, where M / V is the ratio of moment to shear found in an equivalent static or first mode analysis. The height of the end region need not exceed 2 L_w.

11.4.4 *Curvature ductility limitations on the use of singly reinforced walls*

The sectional curvature ductility at the ultimate limit state for walls with a single layer of reinforcement
A2 | shall be less than the values in 2.6.1.3 for a limited ductile plastic hinge region.

11.4.5 *Reinforcement diameters*

A2 | In ductile detailing lengths in a wall, the diameter of bars shall not exceed:

(a) In ductile plastic hinge regions, one tenth of the thickness; and
(b) In limited ductile plastic hinge regions, one eight the wall thickness.

11.4.6 *Transverse reinforcement*

11.4.6.1 *Transverse reinforcement requirements*

The requirements for minimum reinforcement ratio, placing of reinforcement, diameter of transverse bars used and their spacing shall be in accordance with 11.3.11.

11.4.6.2 *Shear reinforcement to be anchored at ends*

Transverse reinforcement shall be provided to resist shear resulting from earthquake forces in accordance with 11.3.10.3.8 and shall be adequately anchored at the wall edges or in boundary elements as required by 7.5.7 for stirrups in beams or by standard hooks as close to the end of the wall as is practicable.

11.4.6.3 *Transverse reinforcement for lateral restraint in plastic hinge regions*

In ductile detailing lengths, longitudinal reinforcement within a wall with two layers of reinforcement, which may be subjected to yielding in compression, and where the longitudinal reinforcement ratio, p_ℓ is computed from: A2

$$p_\ell = \frac{A_s}{ts_v} \quad \text{(Eq. 11–24)}$$

exceeds $2/f_y$ in ductile plastic regions and $3/f_y$ in limited ductile plastic regions, transverse tie reinforcement satisfying the following requirements shall be provided:

(a) Ties suitably shaped shall be so arranged that each longitudinal bar or bundle of bars, placed close to the wall surface, is restrained against buckling by a 90° bend or at least a 135° standard hook of a tie. Where two or more bars at not more than 200 mm centres apart are so restrained, any bars between them are exempted from this requirement;

(b) The area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, shall be computed from Equation 10–41 where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie, including the tributary area of any bars exempted from being tied in accordance with 11.4.6.3(a). Longitudinal bars centred more than 75 mm from the inner face of stirrup ties need not be considered in determining the value of ΣA_b;

(c) The spacing of ties along the longitudinal bars shall not exceed $6d_b$ in ductile plastic regions and $10d_b$ in limited ductile plastic regions, where d_b is the diameter of the longitudinal bar to be restrained.

11.4.6.4 *Transverse reinforcement for lateral restraint of longitudinal bars outside plastic hinge regions*

Outside the plastic hinge regions defined by 11.4.3, transverse reinforcement shall be in accordance with 11.3.

11.4.6.5 *Confinement requirements in plastic hinge region*

Where the neutral axis depth in the potential yield regions of a wall, computed for the appropriate design forces for the ultimate limit state, exceeds:

$$c_c = \frac{0.1\phi_{ow}L_w}{\lambda} \quad \text{(Eq. 11–25)}$$

where

λ = 1.0 for limited ductile regions and

λ = 2.0 for ductile plastic regions as defined by 2.6.1.3 A2

The following requirements shall be satisfied in that part of the wall section which is subjected to compression strains due to the design forces:

(a) Rectangular or polygonal closed hoops, surrounding longitudinal bars, shall be used as in confined columns so that:

$$A_{sh} = \alpha s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07 \right) \quad \text{(Eq. 11–26)}$$

where

α = 0.25 for ductile plastic regions

α = 0.175 for limited ductile plastic regions defined by 2.6.1.3 A2

(b) The length of the confined region of the compressed wall section c' shall be such that:

$$c' \geq c - 0.7\, c_c \quad \text{(Eq. 11–27)}$$

but equal to or greater than $0.5c$.

(c) Longitudinal bars shall be restrained against possible buckling in accordance with 11.4.6.3(a) and (b);

(d) The centre-to-centre spacing of hoops along longitudinal bars in fully ductile plastic regions shall not exceed six times the diameter of the longitudinal bar, or one half of the wall thickness in the confined region. For limited ductile plastic regions the centre-to-centre spacing shall not exceed $10d_b$, or the thickness of the wall in the confined region;

(e) Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following two cases of bars are exempt from this requirement:

(i) Bars or bundles of bars that lie between two laterally supported bars or bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed one-half of the adjacent lateral dimension of the cross section;

(ii) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inside hoops.

A2 | (f) The ductile detailing length of the wall, over which the requirements for hoops in accordance with 11.4.6.5(a) to (d) is to be satisfied, shall be as defined in 11.4.3;

(g) The region to be confined shall contain more than one layer of longitudinal reinforcement.

11.4.7 *Shear strength*

11.4.7.1 *General*

The evaluation of shear strength and the determination of shear reinforcement for walls shall be in
A2 | accordance with 7.5.

11.4.7.2 *Maximum design shear force*

In the estimation of the maximum shear demand on a wall of limited ductility, the maximum shear need not
A2 | exceed that corresponding to the elastic response of the wall element derived using $\mu = 1.25$ and $S_p = 0.9$.

11.4.7.3 *Shear strength provided by the concrete*

In walls, subjected to an axial load N^* the concrete shear strength, V_c, in the end region defined in 11.4.3 shall not exceed:

$$V_c = \left(0.27\lambda\sqrt{f_c'} + \frac{N^*}{4A_g} \right) b_w d \geq 0.0 \quad \text{(Eq. 11–28)}$$

where

$\lambda = 0.25$ for ductile plastic regions

$\lambda = 0.5$ for limited ductile plastic regions defined by Table 2.4

For walls subject to tension, the value of N^* shall be taken as negative and the total nominal shear strength V_n shall not exceed:

A2 | $$V_n \leq \left(\frac{\phi_{ow}}{\alpha} + 0.15 \right) \sqrt{f_c'}\, A_{cv} \leq v_{max}\, A_{cv} \quad \text{(Eq. 11–29)}$$

where

$\alpha = 3.0$ for limited ductile plastic regions

$\alpha = 6.0$ for ductile plastic regions defined by Table 2.4

Linear interpretation between these values may occur when the calculated curvature ductilities lie between the limits provided in Table 2.4 for limited ductile plastic regions and ductile plastic regions.

A2 | v_{max} is given by 7.5.2.

11.4.7.4 *Sliding shear of squat walls*

Squat walls having adequate foundations to enable a plastic hinge to develop at the base shall be designed so as to ensure that no sliding shear failure along the base section could occur before a displacement ductility capacity assigned to such walls can be fully developed.

11.4.8 *Walls with openings*

Openings in structural walls shall be so arranged that unintentional failure planes across adjacent openings, do not reduce the shear or flexural strength of the structure. For ductile cantilever walls with irregular openings appropriate analyses such as based on strut-and-tie models shall establish rational paths for the internal forces. Capacity design procedures shall be used to ensure that the horizontal shear reinforcement will not yield before the flexural strength of the wall is developed.

11.4.9 *Special splice and anchorage requirements*

11.4.9.1 *Splicing of flexural tension reinforcement*

The splicing of the principal vertical flexural tension reinforcement in potential areas of yielding in ductile walls shall be avoided if possible. Not more than one-third in ductile plastic regions, and one-half for limited ductile plastic regions of such reinforcement shall be spliced at the same location where yielding can occur.

11.4.9.2 *Staggering of lapped splices*

The stagger between lapped splices shall be equal to or greater than twice the splice length, and at least one leg of a lateral tie, spaced not further than 10 times the diameter of a longitudinal bar, satisfying the requirements of 8.9.1.2, shall surround lapped bars larger than 16 mm.

11.4.9.3 *Welded and mechanical splices*

Mechanical connections and welded splices satisfying the requirements of 8.7.4.1 may be used in potential areas of yielding in walls, provided that not more than one-half of the reinforcement shall be spliced at one section, and the stagger shall be equal to or greater than 600 mm. Welded splices satisfying 8.7.4.1(a) or mechanical connections meeting the additional testing requirements for stiffness of 8.9.1.3 need not be staggered.

11.4.9.4 *Welded splices in areas where yielding can not occur*

When by capacity design procedure or otherwise it can be shown that yielding of wall reinforcement could not occur, only the requirements of 8.7.5.4 need be satisfied.

NOTES

12 DESIGN OF REINFORCED CONCRETE TWO-WAY SLABS FOR STRENGTH AND SERVICEABILITY

12.1 Notation

a larger side of rectangular contact area, mm
A_s area of non-prestressed tension reinforcement, mm^2
A_v area of shear reinforcement within a distance s, mm^2
b width of compression face, or smaller side of rectangular contact area, mm
b_o perimeter of critical section for slabs and foundations, mm
b_x is the length of the side of the perimeter, b_o, being considered in design for shear reinforcement, mm
b_1 width of critical section defined in 12.7.1(b) measured in the direction of the span for which moments are determined, mm
b_2 width of the critical section defined in 12.7.1(b) measured in the direction perpendicular to b_1, mm
c_1 size of rectangular or equivalent rectangular column, capital, or bracket measured in the direction of the span for which moments are being determined, mm
c_2 size of rectangular or equivalent rectangular column, capital, or bracket measured transverse to the direction of the span for which moments are being determined, mm
d distance from extreme compression fibre to centroid of tension reinforcement, mm
f'_c specified compressive strength of concrete, MPa
f_{yt} lower characteristic yield strength of vertical (stirrup) reinforcement, MPa
h overall thickness of member, mm
h_v total depth of shearhead cross section, mm
L_1 support centre to support centre span of slab not supported by a beam or wall, mm
L_n clear span, in the direction moments are being determined, measured face-to-face of supports, mm
L_s span of slab, mm
L_v length of shearhead arm from centroid of concentrated load or reaction mm
M^* design moment at section at the ultimate limit state, N mm
M_p required plastic moment strength of shearhead cross section, N mm
M_v moment resistance contributed by shearhead reinforcement, N mm
p ratio of tension reinforcement = A_s/bd
p_b value of p for balanced strain conditions derived by 7.4.2.8
s centre-to-centre spacing of shear or torsional reinforcement measured in the direction parallel to the longitudinal reinforcement, mm
t thickness of surfacing and filling material, mm
u larger side of rectangular loaded area allowing for load spread, mm
v smaller side of rectangular loaded area allowing for load spread, mm
v_c shear stress resisted by concrete, MPa
V_c nominal shear strength provided by concrete mechanisms, MPa
V_n nominal shear strength of section, N
v_n total nominal shear stress, MPa
V_s nominal shear strength provided by the shear reinforcement, N
V^* design shear force at section at the ultimate limit state, N
α_s factor accounting for columns
α_v ratio of stiffness of shearhead arm to surrounding composite slab section
β_c ratio of long side to short side of concentrated load or reaction area
η number of arms in shearhead connection
ϕ strength reduction factor (see 2.3.2.2)
θ skew angle

γ_f fraction of unbalanced moment considered to be transferred by flexure

γ_v fraction of unbalanced moment considered to be transferred by eccentricity of shear

12.2 Scope

The provisions of this section shall apply to the design of reinforced concrete two-way slab systems subject predominantly to loading acting at right angles to the plane of the slab.

All references in this section to loads, moments, shear forces and torsions refer to actions at the ultimate limit state unless specifically noted otherwise.

12.3 General

12.3.1 *Slab systems*

A slab system may be supported on columns or walls. If supported by columns, no portion of a column capital shall be considered for structural purposes that lies outside the largest inverted right circular cone or pyramid with a 90° vertex that can be included within the outline of the column capital.

12.3.2 *Floor finishes*

When a separate floor finish is placed on a slab it shall be assumed that:

(a) A floor finish is not included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with the requirements of Sections 13 and 18;

(b) All concrete floor finishes may be taken as part of the required cover or total thickness for non-structural considerations.

12.3.3 *Recesses and pockets*

Solid slabs and slabs with recesses or pockets made by permanent or removable fillers between ribs or joists in two directions are included within the scope of this section.

12.3.4 *Panelled ceilings*

Slabs with panelled ceilings are included within the scope of this section, provided the panel of reduced thickness lies entirely within middle strips, and is equal to or greater than the larger of two-thirds of the thickness of the remainder of the slab, excluding the drop panel, nor less than 100 mm thick.

12.3.5 *Prestressed concrete slabs*

For the design of prestressed concrete slabs refer to Section 19.

12.4 Design procedures

12.4.1 *General*

A slab system may be designed by any procedure satisfying conditions of equilibrium and geometrical compatibility if shown that the design strength is at least that required at the ultimate limit state by either AS/NZS 1170 or other referenced loading standard, and that all serviceability conditions are investigated and satisfied at the serviceability limit state.

12.4.2 *Design methods*

The design moments and shears resulting from distributed or concentrated loads shall be determined using one of the following:

(a) Linear elastic analysis for thin plates as in 12.5.3 and 6.3; or

(b) Non-linear analysis as in 12.5.4 and 6.4; or

(c) Plastic analysis as in 6.5.3 and 12.5.5; or

(d) Idealised frame method of analysis as C6.3.8; or

(e) Simplified method of analysis as in 6.7; or

(f) Empirical method for bridge slabs as in 12.8.2.

12.5 Design for flexure

12.5.1 *General*

The slabs and beams (if any) between supports may be proportioned for the moments at the ultimate limit state prevailing at every section. Design for flexure shall be in accordance with Sections 7 and 9. The range of stresses permitted in the reinforcement due to service live load shall also satisfy the limitation specified under 2.5.2.2 if appropriate.

12.5.2 *Effective area of concentrated loads*

The moments induced in slabs by concentrated loads may take into account the spread of load from the contact area. For a rectangular contact area with sides of length a and b, the sides of the effective rectangular spread shall not exceed the values for u and v given by:

$u = a + 2t + 3h$ .....(Eq. 12–1)

$v = b + 2t + 3h$.....(Eq. 12–2)

Where the load areas derived from Equations 12–1 and 12–2 overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas, but the total width of distribution shall not exceed the total width of the supporting slab.

12.5.3 *Design moments from elastic thin plate theory*

The design bending moments and torsional moments may be determined assuming that the slabs act as thin elastic plates in accordance with 6.3. The assumptions adopted for computing flexural and torsional rigidities of sections shall be consistent throughout the analysis.

12.5.4 *Design moments from non-linear analysis*

The design bending moments and torsional moments may be determined taking into account all relevant non-linear and inelastic effects of the materials in accordance with 6.4.

12.5.5 *Design moments from plastic theory*

The design moments may be determined by a plastic theory such as Johansen's yield line theory or Hillerborg's strip method, provided that the ratios between negative and positive moments used are similar to those obtained by the use of elastic thin plate theory. The maximum value for the tension reinforcement ratio, p, used shall not exceed 0.4 of the ratio producing balanced conditions as defined by 7.4.2.8.

12.5.6 *Slab reinforcement*

12.5.6.1 *Size of drop panels*

Where a drop panel is used to reduce the amount of negative moment reinforcement over the column of a flat slab, the size of drop panel shall be in accordance with the following:

(a) The drop panel shall extend in each direction from centreline of the support a distance equal to or greater than one-eighth of the span length measured from centre-to-centre of supports in that direction;

(b) The projection of the drop panel below the slab shall be at least one-quarter of the slab thickness beyond the drop;

(c) In computing the required slab reinforcement, the thickness of the drop panel below the slab shall not be assumed greater than one-quarter of the distance from the edge of the drop panel to the edge of the column or column capital.

12.5.6.2 *Area of reinforcement*

The area of reinforcement in each direction for two-way slab systems shall be determined from moments at critical sections but shall be equal to or greater than required by 8.8 or more than the limiting value given by the area required to control crack widths as required by 2.4.4.

12.5.6.3 *Spacing of flexural reinforcement*

Spacing of flexural reinforcement shall not exceed the smallest of two times the slab thickness or 300 mm, except for cellular or ribbed construction. In the slab over cellular spaces, or between ribs, the maximum spacing shall be three times the slab thickness.

12.5.6.4 *Extent of positive moment reinforcement at edge*

Positive moment reinforcement perpendicular to a discontinuous supported edge shall extend to the edge of the slab and have embedment, straight or hooked, at least 150 mm in spandrel beams, columns, or walls.

12.5.6.5 *Anchorage of negative moment reinforcement at edge*

Negative moment reinforcement perpendicular to a discontinuous supported edge shall be bent, hooked, or otherwise anchored, in spandrel beams, columns, or walls, to be developed at the face of the support according to the provisions of Section 8.

12.5.6.6 *Anchorage at edge*

Where a slab is not supported by a spandrel beam or wall at a discontinuous edge, or where a slab cantilevers beyond the support, anchorage of reinforcement shall be permitted within the slab.

12.5.6.7 *Reinforcement for torsional moments*

In slabs supported on beams or walls, reinforcement shall be provided in the corners to resist the combined actions due to torsion and flexure found from a rational analyses, or the provisions (a), (b) and (c) shall be satisfied:

(a) Torsional reinforcement shall be provided at any corner where the slab is discontinuous at both edges meeting at that corner. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers, per unit width of slab, shall be at least three-quarters of the area per unit width required for the maximum mid-span positive moment per unit width in the slab;

(b) Torsional reinforcement equal to half that described in (a) shall be provided at a corner contained by edges over only one of which the slab is continuous;

(c) Torsional reinforcement need not be provided at any corner contained by edges over both of which the slab is continuous.

12.5.6.8 *Slabs supported on columns*

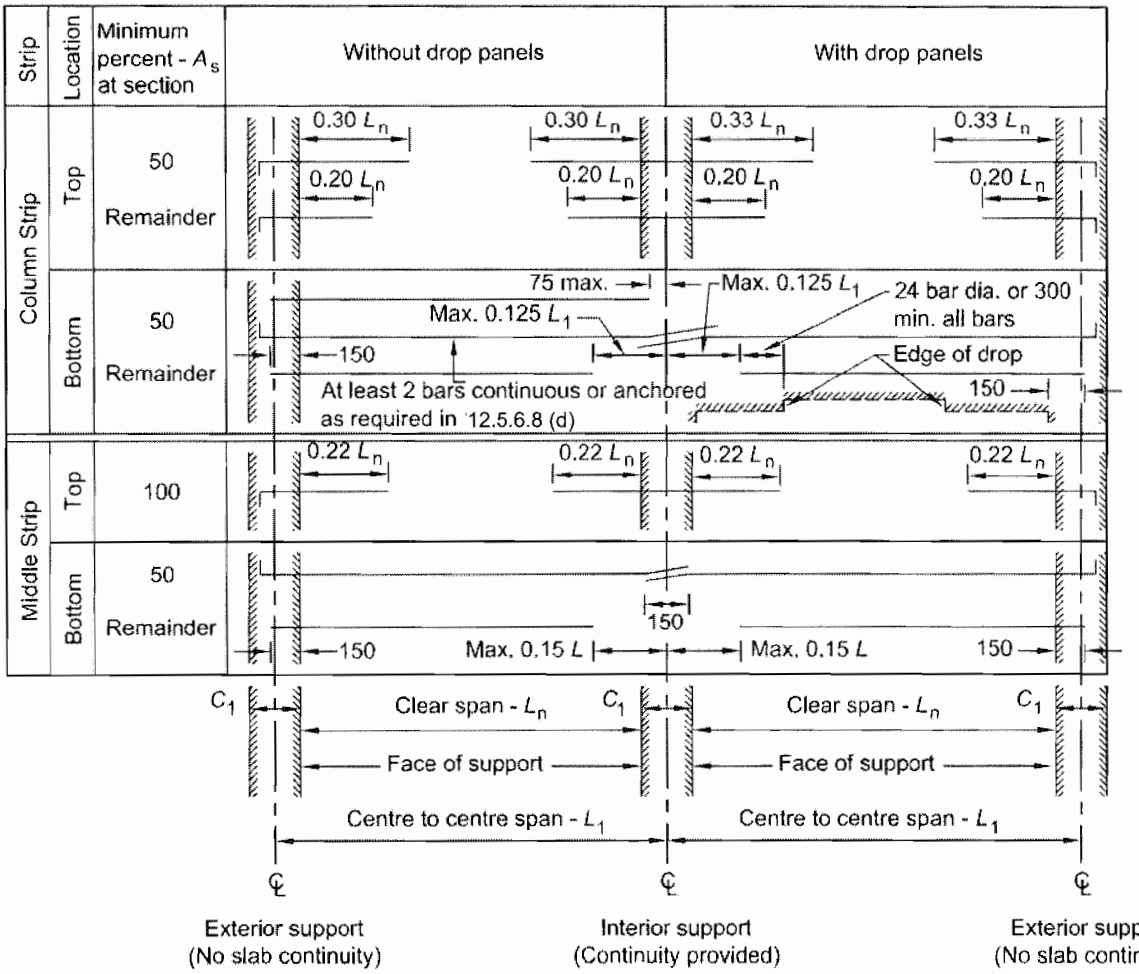
In slabs supported on columns, reinforcement for moments induced by gravity loading shall comply with all the following requirements:

(a) The minimum extensions for reinforcement shall be as prescribed in Figure 12.1;

(b) Where adjacent spans are unequal, extension of negative moment reinforcement beyond the face of the support as prescribed in Figure 12.1 shall be based on requirements of the longer span;

(c) Bent bars shall be used only when the depth-span ratio permits use of bends 45° or less;

(d) Integrity reinforcement shall be provided as required by 12.5.6.9.



NOTE- Refer Figure C6.3 for definition of column and middle strip A2

Figure 12.1 – Minimum extensions for reinforcement in slabs without beams or walls

12.5.6.9 *Integrity reinforcement for slabs supported on columns*

Slabs supported on columns shall satisfy either (a) or (b) as appropriate.

(a) Where slabs are supported on columns reinforcement in the bottom of the slab shall pass through or be anchored in the columns and extend into the slab for a minimum distance of a development length. The area of this reinforcement crossing the interface between the column and the slab, A_{bs}, shall be given by:

$$A_{bs} > \frac{2V^*}{\phi f_y} \quad \text{.....(Eq. 12–3)}$$

(b) In lift slab construction the slab shall be supported on the lower surface by a component or components, which are tied into the column. At least two column strip bottom bars shall be placed in each direction which either pass through the shear head or lifting collar or pass as close to the column as practical. At exterior columns this reinforcement shall be anchored at the shearhead or lifting collar. At all columns this reinforcement shall be extended into the slab beyond collar. At all columns this reinforcement shall be extended into the slab beyond the face of the column for a minimum distance of a development length.

12.6 Serviceability of slabs

12.6.1 *General*

Slabs shall be designed so that the cracking and deflections at the serviceability limit state do not exceed specified limits.

12.6.2 *Cracking*

Flexural cracking slab reinforcement shall comply with the requirements of 12.5.6.2.

12.6.3 *Deflections*

To control deflections the minimum thickness specified in 2.4.3 shall apply unless the calculation of deflection according to 6.8 indicates the lesser thickness may be used without adverse effects.

12.7 Design for shear

12.7.1 *Critical sections for shear*

Shear strength of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

(a) Beam action for the slab or footing, with a critical section perpendicular to the plane of the slab extending across the entire width and located at a distance, d, from the face of the concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with 7.5 and 9.3.9.3 and 9.3.9.4;

(b) Two-way action for a slab or a footing, with a critical section perpendicular to the plane of the slab and located so that its perimeter, b_o, is a minimum, but need not approach closer than $d/2$ to edges or corners of columns, concentrated loads, reaction areas or changes of slab thickness such as edges of capitals or drop panels. For this condition, the slab or footing shall be designed in accordance with 12.7.2 to 12.7.7. For square or rectangular columns, concentrated loads, or reaction areas, the critical sections may have four straight sides. A circular area may be replaced by a square of equal area.

12.7.2 *Design for two-way action*

The design of a slab or footing for two-way action shall be based on 7.5. V_c shall be computed in accordance with 12.7.3.2, V_s shall be computed in accordance with 12.7.4 except that for slabs with shear heads, V_n shall be in accordance with 12.7.5.4. When moment is transferred between slab and column 12.7.7 shall apply.

12.7.3 *Shear strength*

12.7.3.1 *Nominal shear strength for punching shear*

The nominal shear strength for any portion of the critical perimeter, V_n is given by:

$$V_n = V_s + V_c \quad \text{(Eq. 12–4)}$$

Where $V_c = v_c b_o d$ and V_s is given by 12.7.4.

and

$$\frac{V^*}{\phi} \le V_n \quad \text{(Eq. 12–5)}$$

12.7.3.2 *Nominal shear stress resisted by the concrete*

For non-prestressed slabs subject to punching shear the shear stress resisted by the concrete, v_c, shall be the smallest of:

A2 (a) $$v_c = \frac{1}{6} k_{ds} \left(1 + \frac{2}{\beta_c}\right) \sqrt{f'_c} \quad \text{(Eq. 12–6)}$$

where β_c is the ratio of the long side to the short side of the concentrated load or reaction area; or

A2 (b) $$v_c = \frac{1}{6} k_{ds} \left(\frac{\alpha_s d}{b_o} + 1\right) \sqrt{f'_c} \quad \text{(Eq. 12–7)}$$

where α_s = 20 for interior columns, 15 for edge columns, 10 for corner columns; or

(c) $v_c = \frac{1}{3} k_{ds} \sqrt{f'_c}$ .....(Eq. 12–8) A2

where k_{ds} allows for the influence of size on v_c and it is given by $k_{ds} = \sqrt{\frac{200}{d}}$ with the limits of $1.0 \leq k_{ds} \leq 0.5$, where d is the average effective depth round the critical perimeter.

12.7.3.3 *Nominal shear stress, vn for punching shear*

The nominal shear stress for punching shear shall be taken as the sum of:

(a) the shear stress due to the force normal to the slab, as given $V_n/b_o d$

(b) the shear stress due to the transfer of moment to the slab from a column or beam, as given in 12.7.7.

The shear stress shall be based on the perimeter b_o, as defined in 12.7.1(b), with deductions for free edges and openings in the slab as defined in 12.7.6, and the effective depth d.

12.7.3.4 *Maximum nominal shear stress*

The maximum nominal shear stress for punching shear, on any part of the perimeter shall not exceed $0.5\sqrt{f'_c}$.

12.7.3.5 *Shear to be resisted by shear reinforcement for punching shear*

When the nominal shear stress, v_n, on any part of the critical perimeter, b_o, exceeds the critical value of v_c given in 12.7.3.2, the value of v_c round the complete perimeter shall be taken as the smaller of that given by Equations 12–6, 12–7, 12–8 or 12–9. A2

$v_c = \frac{1}{6}\sqrt{f'_c}$ .....(Eq. 12–9)

Shear reinforcement shall be provided to sustain the shear force V_s given by: A2

$V_s = (v_n - v_c)\, b_x d$ .....(Eq. 12–10)

Where b_x is the length of side being considered and d is the effective depth over that length.

12.7.4 ***Shear reinforcement consisting of bars or wires or stirrups***

12.7.4.1 *Design requirements*

Shear reinforcement consisting of effectively anchored bars, wires or single or multiple-leg stirrups is permitted in slabs and footings where the effective depth d is greater than or equal to 150 mm and greater than or equal to 16 times the diameter of the shear reinforcement.

12.7.4.2 *Area of shear reinforcement*

Shear reinforcement required on any side to resist V_s given by Equation 12–10, shall be calculated from appropriate expression below:

(a) Where the shear reinforcement is provided by stirrups, with a yield stress f_{yv}, placed at a spacing s, measured on the perimeter b_o, for a length b_x: A2

$A_v f_{yv} \frac{d}{s} \geq V_s$ and $s \leq \frac{d}{2}$ .....(Eq. 12–11)

(b) Where the shear reinforcement is provided by stirrups or bent up bars, which make an angle of α to the axis of the slab and are spaced at a distance, s:

$A_v f_{yv} (\sin\alpha + \cos\alpha) \frac{d}{s} \geq V_s$ and $s \leq \frac{d}{2}$ .....(Eq. 12–12) A2

For the inclined reinforcement to contribute to A_v, the angle this reinforcement makes to the axis of the member, measured from the direction of decreasing flexural tension, shall be 90° or less.

(c) Where only one line of reinforcement is used.

A2

$$A_v f_{yv} \sin\alpha \geq V_s \quad \text{(Eq. 12–13)}$$

12.7.4.3 *Minimum shear reinforcement for punching shear*

Where shear reinforcement is required over any part of the critical perimeter by 12.7.3.5, shear reinforcement shall not be less than that required to resist a shear force of:

$$V_s = \frac{1}{16}\sqrt{f_c'}\, b_o d \quad \text{(Eq. 12–14)}$$

12.7.4.4 *Placement of shear reinforcement in the form of vertical stirrups*

The distance between the column face and the first line of vertical stirrup legs that surround the column shall not exceed $d/2$. The spacing between adjacent stirrup legs in the first line of shear reinforcement shall not exceed $2d$ measured in a direction parallel to the column face. The spacing between successive lines of shear reinforcement that surround the column shall not exceed $d/2$ measured in a direction perpendicular to the column face.

12.7.4.5 *Anchorage requirements of shear reinforcement in the form of bars or wires*

Slab shear reinforcement in the form of bars or wires shall engage the longitudinal flexural reinforcement in the direction being considered. A 135° stirrup hook shall be used rather than a 90° hook where there is the possibility of cover concrete being lost at the development of the strength of the member. Closed stirrups shall be used in regions that are likely to reach yield stress (ULS). Shear reinforcement consisting of vertical bars anchored at each end with plates having an area of at least 10 times the cross-sectioned area of the bars can be used.

12.7.5 *Shear reinforcement consisting of structural steel I or channel-shaped sections and other equivalent devices*

12.7.5.1 *General*

Shear reinforcement consisting of steel I or channel shapes (shearheads) or other equivalent devices proven by tests to be equally effective may be used in slabs. Provisions of 12.7.5 shall apply where shear due to gravity load is transferred to interior columns. Where moment is transferred 12.7.7.3(c) shall apply.

12.7.5.2 *Details of shearheads*

Details of shearheads shall be as follows:

(a) Each shearhead shall consist of steel shapes fabricated by welding with full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section;

(b) The shearhead shall not be deeper than 70 times the web thickness of the steel shape;

(c) The ends of each shearhead arm may be cut at angles equal to or greater than 30° with the horizontal, provided the plastic moment strength of the remaining tapered section is adequate to resist the shear force attributed to that arm of the shearhead;

(d) All compression flanges of steel shapes shall be located within $0.3d$ of the compression surface of the slab;

(e) The ratio α_v between the flexural stiffness for each shearhead arm and that for the surrounding composite cracked slab section of width ($c_2 + d$) shall be equal to or greater than 0.15;

(f) The plastic moment strength, M_p, required for each arm of the shearhead shall be computed by:

$$M_p = \frac{V^*}{2\phi\eta}\left[h_v + \alpha_v\left(L_v - \frac{c_1}{2}\right)\right] \quad \text{(Eq. 12–15)}$$

where ϕ is the strength reduction factor for flexure, η is the number of arms and L_v is the minimum length of each shearhead arm required to comply with the requirements of 12.7.5.3 and 12.7.5.4.

12.7.5.3 *Critical slab section for shear*

The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters of the distance [L_v – (c_1/2)] from the column face to the end of the shearhead arm. The critical section shall be located so that its perimeter, b_o, is a minimum, but need not be closer than the perimeter defined in 12.7.1(b).

12.7.5.4 *Limit on nominal shear strength*

The nominal shear strength, V_n shall not be taken greater than $0.33\sqrt{f'_c}\ b_o d$ on the critical section defined in 12.7.5.3. When shearhead reinforcement is provided it shall not be taken greater than $0.6\sqrt{f'_c}\ b_o d$ on the critical section defined in 12.7.1(b).

12.7.5.5 *Moment of resistance contributed by shearhead*

A shearhead may be assumed to contribute a moment of resistance, M_v, to each slab column strip computed by:

$$M_v = \frac{\phi\alpha_v V^*}{2\eta}\left(L_v - \frac{c_1}{2}\right) \quad \text{(Eq. 12–16)}$$

where ϕ is the strength reduction factor for flexure, η is the number of arms and L_v is the length of each shearhead arm actually provided. However, M_v shall not exceed the smallest of:

(a) 30 % of the total factored moment required for each slab column strip;

(b) The change in the column strip moment over length L_v;

(c) The value of M_p computed by Equation 12–15;

when unbalanced moments are considered, the shearhead must have adequate anchorage to transmit M_p to the column.

12.7.6 *Openings in slabs*

When openings in slabs are located at a distance less than 10 times the slab thickness from a concentrated load or reaction area, or when openings in flat slabs are located within column strips as defined in Section 6, the critical slab sections for shear defined in 12.7.1(b) and 12.7.5.3 shall be modified as follows:

(a) For slabs without shearheads, that part of the perimeter of the critical section that is enclosed by straight lines projecting from the centroid of the column, concentrated load or reaction area and tangent to the boundaries of the openings shall be considered ineffective;

(b) For slabs with shearheads, the ineffective portion of the perimeter shall be one-half of that defined in (a) above.

12.7.7 *Transfer of moment and shear in slab column connections*

12.7.7.1 *General*

When gravity load, wind or other lateral forces cause transfer of unbalanced moment M^* between a slab and a column, a fraction $\gamma_f M^*$ of the unbalanced moment shall be transferred by flexure in accordance with 12.7.7.2. The remainder of the unbalanced moment given by $\gamma_v M^*$ shall be considered to be transferred by eccentricity of shear about the centroid of the critical section defined in 12.7.1(b) where:

$$\gamma_v = 1 - \gamma_f \quad \text{(Eq. 12–17)}$$

12.7.7.2 *Unbalanced moment transferred by flexure*

The fraction of the unbalanced moment $\gamma_f M^*$ shall be considered to be transferred by flexure within an effective slab width between lines that are one and one-half slab or drop panel thicknesses (1.5h) outside opposite faces of the column or capital, where M^* is the moment to be transferred and:

$$\gamma_f = \frac{1}{1+\frac{2}{3}\sqrt{\frac{b_1}{b_2}}} \quad \text{(Eq. 12–18)}$$

For unbalanced moment about an axis parallel to the edge at exterior supports the value of γ_f by Equation 12–18 may increase up to 1.0 provided that V^* at an edge support does not exceed $0.75\phi V_c$ or at a corner support does not exceed $0.5\phi V_c$. For unbalanced moments at interior supports, and for unbalanced moments about an axis transverse to the edge at exterior supports, the value of γ_f in Equation 12–18 may be increased by 25 % provided that V^* at the support does not exceed $0.4\phi V_c$. The reinforcement ratio p within the effective width defined in 12.7.7.2 shall not exceed $0.375p_b$, where p_b is the balanced reinforcement ratio. Concentration of reinforcement over the column by closer spacing or additional reinforcement shall be used to resist moment on the effective slab width defined in 12.7.7.2.

12.7.7.3 *Unbalanced moment transferred by eccentricity of shear*

A fraction of the unbalanced moment $\gamma_v M^*$, considered to be transferred by eccentricity of shear results in a shear stress which shall be assumed to vary linearly about the centroid of the critical section defined in 12.7.1(b). The maximum shear stress due to the design shear force V^* and moment $\gamma_v M^*$ shall not exceed ϕv_n, where:

(a) For members without shear reinforcement:

$$\phi v_n = \frac{\phi V_c}{b_o d} \quad \text{(Eq. 12–19)}$$

(b) For members with shear reinforcement other than shearheads:

$$\phi v_n = \frac{\phi(V_c + V_s)}{b_o d} \quad \text{(Eq. 12–20)}$$

where V_c and V_s are defined in 12.7.3.2, 12.7.3.5 and 12.7.4. The design shall take into account the variation of shear stress around the column. The shear stress shall not exceed $0.17\sqrt{f'_c}$ at the critical section located $d/2$ outside the outermost line of stirrup legs that surround the column;

(c) When shear reinforcement consisting of shearheads is used the sum of the shear stresses due to vertical load acting on the critical section defined by 12.7.5.3 and the shear stress resulting from moment transferred by eccentricity of shear about the centroid of the critical section defined in 12.7.1(b) shall not exceed $\phi\, 0.33\sqrt{f'_c}$.

12.8 Design of reinforced concrete bridge decks

12.8.1 *Design methods*

Two methods of design may be used for reinforced concrete bridge deck slabs supported on beams or girders.

(a) Empirical design based on assumed membrane action, in accordance with 12.8.2; or

(b) Elastic plate bending analysis in accordance with 12.8.3;

where the dimensional and structural limitations of the empirical design method are not met, or for deck cantilevers, the elastic plate bending analysis design method shall be used.

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12.8.2 *Empirical design based on assumed membrane action*

12.8.2.1 *General*

Slabs satisfying the requirements below and designed in accordance with this method need not be analysed for bending moments and shears in the slab due to traffic loading, and the requirements of Section 2 and 9 shall be waived. This method is not applicable to cantilevered slabs.

12.8.2.2 *Conditions*

The empirical design method shall only be used if all of the following conditions are satisfied:: A2

(a) The supporting components are made of steel or concrete;

(b) The deck is fully cast-in-place and water cured;

(c) The deck is of uniform depth except for haunches at girder flanges and other local thickening;

(d) The deck is made composite with the supporting structural components. In the case of the negative moment region of continuous steel girders, two or more shear connectors at a spacing of less than 600 mm shall be provided;

(e) Cross-frames or diaphragms are used throughout the cross section at lines of support;

(f) For cross sections involving torsionally stiff units such as individual separated box beams, either intermediate diaphragms between the boxes are provided at a spacing not exceeding 8.0 m, or the need for supplemental reinforcement over the webs to accommodate transverse bending between the box units is investigated, and reinforcement is provided if necessary;

(g) The ratio of the span length, L_s, to design depth does not exceed 18.0 and is not less than 6.0, where the design depth is the slab thickness excluding any sacrificial wearing surface;

(h) The span length, L_s, as specified in 12.8.4 does not exceed 4.100 m;

(i) The minimum slab thickness is equal to or greater than 175 mm, excluding any sacrificial wearing surface;

(j) The core depth of the slab is not less than 100 mm. The core depth is defined as the slab thickness less any wearing surface and the top and bottom cover thicknesses;

(k) There is an overhang beyond the centreline of the outside girder of at least five times the slab thickness. This condition may be considered satisfied if the overhang is at least three times the slab thickness and a structurally continuous concrete kerb or barrier is made composite with the overhang;

(l) The specified 28 day strength of the deck concrete is equal to or greater than 30 MPa;

(m) The design vehicle axle and wheel loads being no greater in their effects than those imposed by the Transit Bridge Manual design vehicle loadings (as at August 2008).

12.8.2.3 *Reinforcement*

For slabs meeting the above conditions, the deck reinforcement shall comprise:

(a) Layers of reinforcement in two directions at right angles in the top and bottom of the slab, placed as close to the outside surfaces as possible, as permitted by cover requirements;

(b) The reinforcing steel shall have a yield strength greater than or equal to 420 MPa;

(c) The minimum amount of reinforcement shall be 570 mm^2/m of steel in each direction in the bottom layer, 380 mm^2/m of steel in each direction in the top layer;

(d) All reinforcement shall be straight bars except that hooks may be provided where required;

(e) The maximum spacing of the reinforcement may be 300 mm;

(f) The bars shall be spliced by lapping or by butt welding, or by mechanical connections satisfying 8.7.5.2 only;

(g) For skew angles, θ, greater than 25°, the specified reinforcement in both directions shall be doubled in the end regions of the deck. The span end regions are as defined in Figure 12.2.

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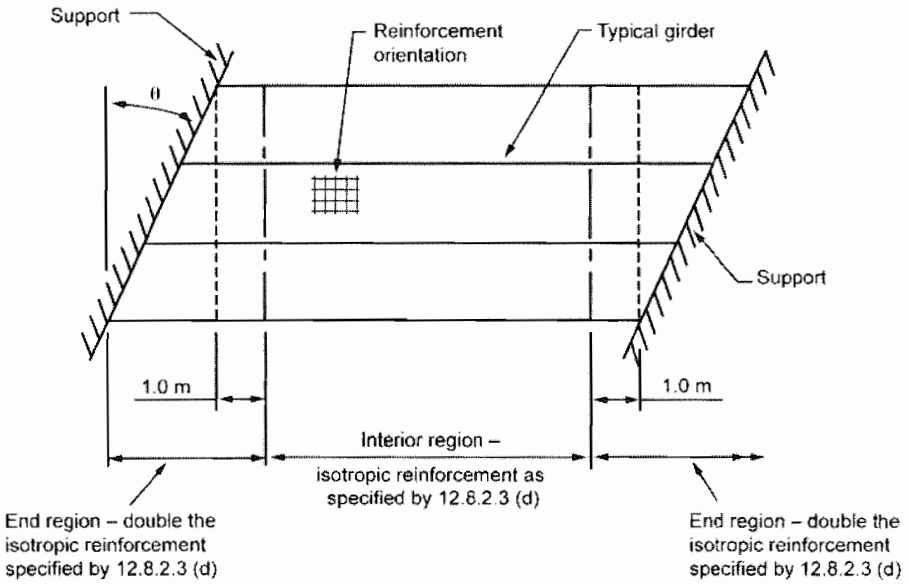

Support
Reinforcement orientation
Typical girder
θ
Support
1.0 m
1.0 m
Interior region – isotropic reinforcement as specified by 12.8.2.3 (d)
End region – double the isotropic reinforcement specified by 12.8.2.3 (d)
End region – double the isotropic reinforcement specified by 12.8.2.3 (d)

Figure 12.2 – Reinforcement of skewed slabs by the empirical method

12.8.2.4 *Longitudinal negative moments in continuous structures*

The longitudinal bars of the isotropic reinforcement may participate in resisting negative moments at an internal support in a continuous structure.

12.8.2.5 *Minimum slab thickness*

For deck slabs designed by the empirical method of 12.8.2, the minimum slab thickness requirements of 12.8.2 shall take precedence over the other requirements of NZS 3101.

12.8.3 *Design based on elastic plate bending analysis*

12.8.3.1 *Determination of moments*

The moments in deck slabs due to the local effects of wheels shall be determined by an elastic analysis, assuming the slab to act as a thin plate. Adequate allowance shall be made for the effects of the rotation of the edges monolithic with beams, due to torsional rotation of the beams, and the effects of relative displacement of beams.

12.8.3.2 *Deck slab also functioning as a flange*

Where the deck slab resists the effects of live load by the top flange of a box girder also functioning as the deck slab, or a transverse distribution member integral with the deck slab the slab, shall be designed for the sum of the effects of the appropriate loading for each condition.

12.8.3.3 *Haunched slabs*

Where slabs are haunched at fixed edges, allowance for the increase in support moment due to the haunch shall be made either by modifying the moments determined for slabs of uniform thickness, or by a rational analysis that takes into account the varying section.

A2

12.8.4 *Span length of reinforced concrete bridge deck slabs*

12.8.4.1 *General*

The following requirements apply to the determination of slab span length, L_s, as used in Table 2.3 and in the application of the empirical design method specified in 12.8.2.

12.8.4.2 *Slab span for a uniform slab monolithic with webs*

For a uniform concrete slab, monolithic with concrete webs, the slab span length, L_s, shall be taken as the clear span.

12.8.4.3 *Span length for a haunched slab*

For a slab monolithic with concrete webs or tied down to steel girders locally haunched adjacent to its supports, where the thickness at the root of the haunch is at least 1.5 times the thickness at the centre of

the slab and the length of the haunch is less than twice the slab thickness, the slab span length, L_s, shall be taken as the distance between the mid-points of opposite haunches. A2

12.8.4.4 *Span length of slab on steel girders*

For a uniform slab on steel girder, the slab span, L_s, shall be taken as the average of the distance between webs and the clear distance between flange edges.

12.8.4.5 *Span length of slab spanning between non-uniformly spaced supports*

Where the spacing of supporting components varies, the span length, L_s, shall be taken as the larger of the deck lengths at the two locations shown in Figure 12.3.

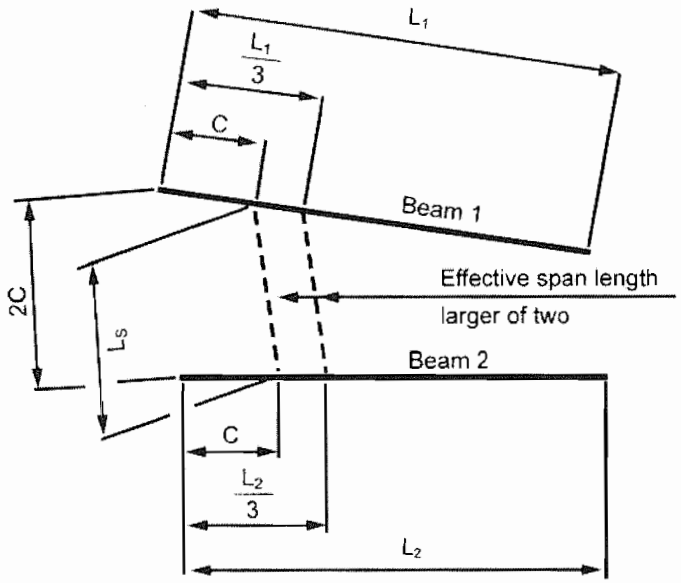


Figure 12.3 – Effective span length for non-uniform spacing of beams

NOTES

13 DESIGN OF DIAPHRAGMS

13.1 Notation

A_{cv} area of concrete section resisting shear, mm^2
f'_c specified compressive strength of concrete, MPa
V_c nominal shear strength provided by concrete, N
V_n total nomination shear strength of section, N

13.2 Scope and definitions

Provisions of this section apply to diaphragms in buildings. They are defined as relatively thin but stiff horizontal or nearly horizontal structural systems which transmit in-plane lateral forces to, or between lateral force-resisting elements.

Diaphragms which are not designed to dissipate energy at the ultimate limit state shall meet the requirements of 13.3. Diaphragms designed to dissipate energy shall meet the requirements of 13.3 as modified by 13.4.

13.3 General principles and design requirements

13.3.1 *Functions of diaphragms*

Diaphragms may be required to function simultaneously as floors subjected to gravity loads and as diaphragms to transfer in-plane actions due to lateral forces.

13.3.2 *Analysis procedures*

Rational analysis shall be used to establish that there is adequate in-plane flexural and shear strength at the ultimate limit state. Flexural and shear stiffnesses of diaphragms and effects of creep, shrinkage and thermal gradient shall be considered at the serviceability limit state.

13.3.3 *Openings*

Penetrations of diaphragms by openings shall not impair the feasible transmission of internal forces. Analysis and design shall be based on strut-and-tie models simulating admissible and effective in-plane load paths between and around openings.

13.3.4 *Stiffness*

Analysis for the internal forces transmitted between diaphragms and their supports shall account for the stiffness of the chosen load path as dictated by the presence of openings.

13.3.5 *Reinforcement shall be anchored*

Reinforced concrete slabs cast with the supporting beams, columns or walls designed to carry gravity loads in one-way or in two-ways in accordance with Section 12 shall be reinforced in two orthogonal directions with an amount in each direction equal to or greater than that required by 8.8. For diaphragm actions such reinforcement shall be developed beyond the edges of slab panels within boundary beams or walls.

13.3.6 *Changes in depth*

Where changes in the depth of the diaphragm are provided, the impact of this on force transfer shall be considered.

13.3.7 *Diaphragms incorporating precast concrete elements*

13.3.7.1 *Composite concrete flexural members*

Where composite action of precast concrete floor elements and a cast-in-place concrete topping is relied on, the requirements of 18.5 shall be satisfied.

13.3.7.2 *Requirements for toppings transferring diaphragm forces*

A cast-in-place reinforced concrete topping over precast floor systems may be used to transfer diaphragm forces, provided that:

(a) The cast-in-place concrete topping is at least 50 mm thick; and

(b) Minimum reinforcement in two principal directions in accordance with 8.8 is placed in the topping slab; and

(c) Class E reinforcing bars to AS/NZS 4671 are provided around the perimeter of the floor span in accordance with 13.3.7.3; and

(d) Either:

  (i) The requirements of 18.5.4.1 relating to the interface between the *in situ* topping and precast units are satisfied; or

  (ii) Connections between precast elements and the cast-in-place topping are provided in accordance with 13.4.3.

13.3.7.3 *Starter and continuity bars*

At the perimeter of the floor, Class E starter bars to AS/NZS 4671 shall be provided to anchor the topping to the supporting element.

At interior supports, where the floor is continuous over the supports, Class E continuity bars shall be provided in the topping above and perpendicular to the supporting member.

The required area and the length of the this reinforcement shall be determined by analysis but shall have a capacity in excess of 100 kN/m and extend into the topping beyond the end/edge of the precast by at least 600 mm. The curtailment of these bars shall be staggered to ensure that no more than 50 % of the bars are curtailed at the same location.

13.3.7.4 *Transfer of diaphragm forces across joints in untopped systems*

Diaphragm action of precast and cast-in-place systems without an effective cast-in-place concrete topping shall be assumed only if the transfer of in-plane forces across appropriately formed joints between concrete components, consistent with diaphragm action in both principal axes of the structural system, is equivalent to that of a cast-in-place concrete slab with reinforcement satisfying at least the requirements of 8.8.

13.3.7.5 *Connection of diaphragm to primary lateral force-resisting system*

Connections by means of reinforcement from precast or cast-in-place concrete diaphragms to components of the primary force-resisting systems shall be adequate to accommodate the expected deformations at the interface while maintaining load paths.

13.3.8 *Reinforcement detailing for elastically responding diaphragms*

When diaphragms are designed for earthquake forces in accordance with NZS 1170.5, or other referenced loading standard, no special requirements for detailing of reinforcement for ductility need be satisfied.

13.3.9 *Strength of diaphragms in shear*

The strength design of diaphragms for shear shall be based upon strut and tie models in accordance with Appendix A.

13.3.10 *Columns to be tied to diaphragms*

Columns are to be tied to diaphragms in accordance with 10.3.6.

13.4 Additional design requirements for elements designed for ductility in earthquakes

13.4.1 *Design forces for designed to dissipate energy diaphragms*

Diaphragms designed to dissipate energy from earthquake induced forces shall only be permitted when justified by special theoretical and experimental studies.

13.4.2 *Reinforcement detailing*

In diaphragms designed as permitted by 13.4.1, inelastic regions must be clearly identified and appropriate detailing of the reinforcement corresponding to the relevant requirements of Sections 7 and 11 shall be provided.

13.4.3 *Diaphragms incorporating precast concrete elements*

13.4.3.1 *Precast shall be tied to topping*

In potential plastic hinge regions, the consequences of delamination of the topping from the precast members shall be assessed. Where composite action is required to support $G + \Psi_c Q$ as defined in AS/NZS 1170 Part 0, connectors shall be provided between the topping and precast to satisfy:

(a) Ties with an effective area of 40 mm^2 per m^2 of floor area, or equivalent connectors, shall connect the topping to the precast element;

(b) Spacing of connectors shall not exceed 1500 mm, and the tributary area of topping reliant on each connector shall not exceed 2.25 m^2;

(c) Connectors shall engage horizontal reinforcement, or shall be otherwise effectively anchored into both the topping and the precast element, or into the joints between precast elements where contact surfaces are in accordance with 18.5.4.1.

NOTES

14 FOOTINGS, PILES AND PILE CAPS

14.1 Notation

A_g gross area of section, mm^2
A_s area of non-prestressed reinforcement, mm^2
b width of compression face of member, mm
d distance from extreme compression fibre to centroid of tension reinforcement, mm
d_p diameter or side dimension of pile at footing base, mm
f'_c specified compressive strength of concrete, MPa
f_y lower characteristic yield strength of non-prestressed reinforcement, MPa
p_t ratio of non-prestressed tension reinforcement A_s/bd
β_c ratio of long side to short side of footing

14.2 Scope

The provisions of this section shall apply for the structural design of isolated and combined footings. Basic principles for the structural design of piles are also included.

Footings, piles and pile caps containing plastic regions with material strain demands less than or equal to the limits for nominally ductile plastic regions defined in Table 2.4 shall meet the requirements of 14.3. Footings, piles and pilecaps which are designed for ductility in response to earthquake effects shall meet the requirements of 14.3 as modified by 14.4.

14.3 General principles and requirements

14.3.1 *Serviceability and ultimate limit state design*

The base area of footings or the number and arrangement of piles shall be determined from the greater of:

(a) The external forces and moments resulting from ultimate limit state loads (transmitted by the foundation element to the soil or piles) and ultimate soil pressure or the ultimate pile capacity selected through principles of soil mechanics; or

(b) The footing area or number and arrangement of piles necessary to ensure overall and differential settlement criteria are met at the serviceability limit state.

14.3.2 *Design of pile caps*

Pile caps shall be designed using either flexural theory or a strut-and-tie approach.

14.3.3 *Moment in footings*

14.3.3.1 *Moment on a section*

The moment on any section of a footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over the entire area of footing on one side of that vertical plane.

14.3.3.2 *Critical design section*

The maximum design moment for an isolated footing shall be computed as prescribed in 14.3.3.1 at critical sections located as follows:

(a) At the face of a column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;

(b) Halfway between the middle and edge of a wall, for footings supporting a masonry wall;

(c) Two times the base plate thickness out from the column face for a footing supporting a column with an unstiffened base plate;

(d) By rational analysis for a column supported on a base plate with stiffeners.

14.3.3.3 *Strength of footings in flexure*

Footings that resist imposed actions as beams or one-way slabs, shall be designed based on the assumptions of 7.4 and 9.3.6 and 9.3.8.

Footings that resist imposed actions as two-way slabs, shall be designed in accordance with 12.5.

14.3.3.4 *Foundation elements supporting circular or regular polygon shaped columns or pedestals*

Circular or regular polygon shaped concrete columns or pedestals may be treated as square members of the same area for determining the location of critical sections for moment, shear, and development of reinforcement in foundation or elements.

14.3.4 *Shear in footings*

14.3.4.1 *General*

The shear design of footings that resist imposed actions as beams or one-way slabs, shall be designed based on the assumptions of 9.3.9 and 12.7.

The shear design of footings that resist imposed actions as two-way slabs, shall be designed in accordance with 12.7.

14.3.4.2 *Spread footings and footing supported by piles*

The location of the critical section for shear in accordance with 7.5 shall be measured from the face of a column, pedestal, or wall, for footings supporting a column, pedestal, or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from the location defined in 14.3.3.2 (c) and (d).

14.3.4.3 *Shear in pile caps*

The computation of shear on any section through a pile cap shall be in accordance with the following:

(a) The entire reaction from any pile whose centre is located d_p /2 or more outside the section shall be considered as producing shear on that section;

(b) The reaction from any pile whose centre is located d_p /2 or more inside the section shall be considered as producing no shear on that section;

(c) For intermediate positions of pile centre, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between the full value at d_p/2 outside the section and zero value at d_p/2 inside the section.

14.3.5 *Development of reinforcement in footing*

14.3.5.1 *General*

Detailing for the development of reinforcement in footing shall be in accordance with Section 8.

14.3.5.2 *Development of reinforcement*

The calculated tension or compression in reinforcement at each section shall be developed on each side of that section by sufficient embedment length, end anchorage, hooks (tension only), or a combination thereof, and in the case of mesh, by overlapping grids.

14.3.5.3 *Critical sections for development*

Critical sections for the development of reinforcement shall be assumed at the same locations as defined in 14.3.3.2 for maximum design moment, and at all other vertical planes where changes of section or reinforcement occur.

14.3.5.4 *Curtailment of reinforcement*

Curtailment of flexural reinforcement shall be in accordance with 8.6.12, 8.6.13 and 8.6.14.

14.3.6 *Piled foundations*

14.3.6.1 *General*

The design of piles shall in addition to the requirements of Section 2, include due consideration of the loads associated with installation of the piles.

14.3.6.2 *Strength of piles in axial load and flexure*

Reinforced concrete piles shall be designed for axial load and flexure in accordance with 7.4.

Prestressed concrete piles shall be designed for axial load and flexure in accordance with 19.3.6 and 19.3.7.2.

14.3.6.3 *Details for upper ends of piles*

It shall be assumed that plastic hinges form in the upper ends of piles, except where it can be established that movement of the structure relative to the ground, or ground deformation, will not cause yielding of the longitudinal reinforcement. Such potential plastic hinges shall be reinforced as potential plastic hinge regions.

14.3.6.4 *Longitudinal reinforcement in reinforced concrete piles*

In regions where yielding of the reinforcement is expected at the ultimate limit state, and over a length defined by 14.3.6.10, the minimum amount of reinforcement, and detailing, shall be as specified by 10.3.8.

In regions where yielding of the reinforcement is not expected at the ultimate limit state, the minimum amount of reinforcement shall be as specified by 14.3.6.5.

14.3.6.5 *Minimum longitudinal reinforcement in reinforced concrete piles*

The minimum longitudinal reinforcement ratio, p_t, in piles shall be as follows:

(a) For piles having a gross area of section, A_g, equal to, or less than 0.5 x 10^6 mm^2, p_t shall be equal to or greater than 2.4 /f_y;

(b) For piles having a cross-sectional area, A_g, equal to or greater than 2 x 10^6 mm^2, p_t shall be equal to or greater than 1.2/f_y;

(c) For piles having a cross-sectional area, A_g, between 0.5 x 10^6 mm^2, and 2 x 10^6 mm^2, p_t shall be equal to or greater than given by Equation 14–1;

$$p_t = \frac{2400}{f_y\sqrt{2A_g}} \quad \text{.....(Eq. 14–1)}$$

14.3.6.6 *Maximum longitudinal reinforcement in reinforced concrete piles*

The area of longitudinal reinforcement in piles at any location including lap splices shall be less than 0.08 times the gross area, A_g. | A1

14.3.6.7 *Longitudinal reinforcement in prestressed concrete pile*

Members with average prestress f_{ps} less than 1.5 MPa shall have minimum reinforcement in accordance with 14.3.6.4.

In regions where yielding of the reinforcement is expected at the ultimate limit state, and over a length defined by 14.3.6.10, the minimum amount of reinforcement and detailing shall be as specified by 19.4.4.1 to 19.4.4.4.

14.3.6.8 *Strength of piles in shear*

Reinforced concrete piles shall be designed for shear based on the assumptions of 7.5 and 10.3.10. For piles smaller than 250 mm square or circular, the minimum shear reinforcement requirements of 10.3.10.4.4 may be waived if the design shear force, V^*, is less than one-half of the shear strength provided by the concrete (ϕV_c).

Prestressed concrete piles shall be designed for shear in accordance with 19.3.11. For piles smaller than 250 mm square or circular, the minimum shear reinforcement requirements of 10.3.10.4.4 may be waived if the design shear force, V^*, is less than one-half of the shear strength provided by the concrete (ϕV_c).

14.3.6.9 *Piled foundations with permanent casing*

For piled foundation systems the permanent shell or casing of a pile may be considered as providing a proportion of the strength of the pile. For steel casings an appropriate allowance shall be made for loss of wall thickness by corrosion during the specified intended life of the structure.

14.3.6.10 *Transverse reinforcement for confinement and lateral restraint of longitudinal bars*

Where yielding of the longitudinal reinforcement is expected, transverse reinforcement complying with 10.3.10.5 for circular piles, and 10.3.10.6 for square piles shall be provided over the length defined by the greater of :

(a) The length defined in 10.4.5 plus three pile diameters, or three times the section depth;

(b) Twice the length defined in 10.4.5.

14.4 Additional design requirements for members designed for ductility in earthquakes

14.4.1 *Designing for ductility*

14.4.1.1 *General*

The foundation system shall maintain its ability to support the design gravity loads while sustaining the chosen earthquake energy dissipating mechanisms in the structure at the development of the relevant overstrength actions of the structure.

14.4.1.2 *Compliance with additional requirements*

All members shall comply with the additional requirements for members designed for seismic forces as set down in the relevant sections of this Standard. However, flexural members, other than piles, which have a nominal strength greater than the greatest total seismic action that can be transmitted to them from the superstructure, need not comply with these requirements.

14.4.1.3 *Longitudinal reinforcement*

Within the region defined by 14.3.6.10, longitudinal reinforcement for reinforced concrete piles shall comply with 10.4.6.

For prestressed concrete piles, longitudinal reinforcement shall comply with19.4.4.1 to 19.4.4.3 within the region defined by 14.3.6.10.

14.4.1.4 *Transverse reinforcement*

Within the region defined by 14.3.6.10, transverse reinforcement for reinforced concrete piles shall comply
A1 | with 10.4.7.

For prestressed concrete piles, transverse reinforcement shall comply with 19.4.4.4 and 19.4.4.5 within the region defined by 14.3.6.10.

14.4.2 *Pile caps*

Where earthquake induced moments are to be transmitted at the intersection of columns and pile caps, design of this region as a beam column joint shall be in accordance with 15.4.

15 DESIGN OF BEAM COLUMN JOINTS

15.1 Notation

A_g gross area of column section, mm^2

A_{jh} total area of effective horizontal joint shear reinforcement in the direction being considered, mm^2

A_{jv} total area of effective vertical joint shear reinforcement, mm^2

A_s area of non-prestressed tension beam reinforcement including bars in effective tension flanges, where applicable, mm^2

A_s' area of non-prestressed compression reinforcement, mm^2

A_s^* greater of the area of top or bottom beam reinforcement passing through a joint, mm^2

b_c overall width of column, mm

b_j effective width of joint, mm (see 15.3.4)

b_w web width, mm

C_j $\dfrac{V_{jh}}{V_{jx}+V_{jz}}$

d_b normal diameter of longitudinal reinforcing bar, mm

e eccentricity between the centrelines of the webs of a beam and a column at a joint, mm

f_c' specified compressive strength of concrete, MPa

f_s computed steel tensile stress, MPa

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

f_{yh} lower characteristic yield strength of horizontal joint shear reinforcement, MPa

f_{yv} lower characteristic yield strength of non-prestressed vertical joint shear reinforcement, MPa

h_b overall depth of beam, mm

h_c overall depth of column in the direction of the horizontal shear to be considered, mm

$\backslash P_{cs}$ force after all losses in prestressing steel that is located within the central third of the beam depth, N

N^* design axial column load at ultimate limit state, N

N_o^* minimum design axial column load at the ultimate limit state, consistent with capacity design principles where relevant and including vertical prestressing where applicable, taken positive when causing compression occurring simultaneously with V_{jh}, N

V_{ch} nominal horizontal shear force transferred across a joint by the diagonal compression strut mechanism, N

V_{cv} nominal vertical shear force transferred across a joint by the diagonal compression strut mechanism, N

V_{jh} nominal horizontal shear force transferred across a joint in the direction being considered, N

V_{jx} nominal horizontal joint shear force transferred in x direction, N

V_{jv} nominal vertical shear force transferred across a joint, N

V_{jz} nominal horizontal joint shear force transferred in z direction, N

V_{sh} nominal horizontal shear force transferred across a joint by the truss mechanism, N

V_{sv} nominal vertical shear force transferred across a joint by the truss mechanism, N

V_{jh}^* design horizontal shear force across a joint, N

V_{cjh}^* design horizontal shear force across a joint at overstrength, N

V_{jv}^* design vertical shear force across a joint, N

α_i factor for determining V_{ch}

α_v factor for determining V_{cv}

β ratio of area of compression beam reinforcement to that of the tension beam reinforcement at exterior beam column joint, not to be taken larger than unity

ΔA_{jh} permitted reduction in horizontal joint shear reinforcement, mm^2

15.2 Scope

15.2.1 *General*

A2 Provisions of this section apply to design of beam column joints subject to shear induced by gravity loads or earthquake forces or both, a minimum of 70 % of the top beam reinforcement and 70 % of the bottom beam reinforcement contributing to nominal flexural strength either pass through the column or are anchored in the column. Where this does not apply, design shall be based on strut and tie analysis.

Where all regions immediately adjacent to a beam column joint remain elastic or contain nominally ductile plastic regions, the joint zone shall be designed to meet the requirements of 15.3. Where any of the members framing into the joint zone contain ductile or limited ductile plastic regions adjacent to the joint zone, it shall be designed to meet the requirements of 15.3 as modified by 15.4. The written requirements take precedence over Table C15.1.

15.2.2 *Alternative methods*

In lieu of the methods specified in 15.3 and 15.4, principles of mechanics based on strut-and-tie models or equivalent may be used to determine the internal forces and hence the required shear reinforcement and anchorages in beam column joints.

15.3 General principles and design requirements for beam column joints

15.3.1 *Design criteria*

Beam column joints shall satisfy the following criteria:

(a) At the serviceability limit state, a joint shall perform at least as well as the members that it joins;

(b) At the ultimate limit state, a joint shall have a design strength sufficient to resist the most adverse load combinations sustained by the adjoining members, as specified by AS/NZS 1170 or other referenced loading standard.

15.3.2 *Design forces*

The design forces resulting from gravity loads and wind forces acting on a beam column joint shall be evaluated from the maximum internal forces introduced by all members meeting at the joint, subjected to the most adverse combination of ultimate limit state loads as required by AS/NZS 1170 or other referenced loading standard, with the joint in equilibrium. The design joint shear force for seismic load cases where nominally ductile plastic regions are expected to form adjacent to the joint, shall be calculated assuming the reinforcement in the plastic region yields, (f_y).

15.3.3 *Consideration of concurrency*

Where beams frame into the joint from two directions, these forces need only be considered in each
A2 direction independently.

15.3.4 *Maximum horizontal joint shear force*

The horizontal design shear force across a joint, V_{jh}^{*}, shall not exceed the smaller of $0.20\, f'_c b_j h_c$, or $10 b_j h_c$ where h_c is the overall depth of the column in the direction of the horizontal shear to be considered and the effective joint width, b_j, shall be taken as:

A2 (a) Where $b_c \geq b_w$:
either $b_j = b_c$, or $b_j = b_w + 0.5\, h_c$, whichever is the smaller;

(b) Where $b_c < b_w$:
either $b_j = b_w$, or $b_j = b_c + 0.5\, h_c$, whichever is the smaller.

15.3.5 *Design principles, mechanisms of shear resistance*

The joint shear shall be assumed to be resisted by a concrete mechanism and a truss mechanism, comprising horizontal and vertical stirrups or bars and diagonal concrete struts.

Superposition of the two mechanisms for horizontal and vertical joint shear transfer results in nominal shear forces being transferred across the joint core as follows:

$V_{jh} = V_{ch} + V_{sh} = V_{ch} + A_{jh} f_{yh}$ .....(Eq. 15–1)

$V_{jv} = V_{cv} + V_{sv} = V_{cv} + A_{jv}f_{yv}$ .....(Eq. 15–2)

Where V_{ch} and V_{cv} are the nominal horizontal and vertical shear forces transferred across the joint core by the diagonal compression strut mechanism respectively, and V_{sh} and V_{sv} are the nominal horizontal and vertical shear forces transferred across the joint core by the truss mechanism, across the corner to corner potential diagonal failure plane, respectively.

15.3.6 *Horizontal joint shear reinforcement*

15.3.6.1 *Design basis for horizontal shear*

The design for horizontal shear force is based on:

$$V_{jh}^{*} \le \phi V_{jh} \quad \text{.....(Eq. 15–3)}$$

15.3.6.2 *Area of horizontal joint shear reinforcement*

The area of total effective horizontal joint shear reinforcement corresponding with each direction of horizontal joint shear force shall be:

$$A_{jh} = \frac{V_{jh}^{*} - \phi V_{ch}}{\phi f_{yh}} \quad \text{.....(Eq. 15–4)}$$

where

$$\phi V_{ch} = V_{jh}^{*}\left(0.5 + \frac{C_j N^{*}}{A_g f_c'}\right) \quad \text{.....(Eq. 15–5)}$$

A1 | The distribution of this reinforcement within the joint shall be as required by 15.4.4.3.

15.3.7 *Vertical joint shear reinforcement*

15.3.7.1 *Design basis for vertical shear*

The design for vertical shear force is based on:

$$V_{v}^{*} \le \phi V_{jv} \quad \text{.....(Eq. 15–6)}$$

15.3.7.2 *Area of vertical joint shear reinforcement*

The area of total effective vertical joint shear reinforcement corresponding with each direction of horizontal joint shear force shall be:

$$A_{jv} = \frac{V_{jh}^{*}\dfrac{h_b}{h_c} - \phi V_{cv}}{\phi f_{yv}} \quad \text{.....(Eq. 15–7)}$$

where

$$\phi V_{cv} = 0.6 V_{jh}^{*}\frac{h_b}{h_c} + C_j N^{*} \quad \text{.....(Eq. 15–8)}$$

The distribution of this reinforcement within the joint shall be as required by 15.4.5.2.

15.3.8 *Confinement*

The horizontal transverse confinement reinforcement in beam column joints shall be equal to or greater than that required by 10.3.10.5 and 10.3.10.6, with the exception of joints connecting beams at all four column faces in which case the transverse joint reinforcement may be reduced to one-half of that required in 10.3.10.5 and 10.3.10.6. In no case shall the stirrup-tie spacing in the joint core exceed 10 times the diameter of the smallest column bar or 200 mm, whichever is less.

15.4 Additional design requirements for beam column joints with ductile, including limited ductile, members adjacent to the joint

15.4.1 *General*

Special provisions are made in this section for beam column joints that are subjected to forces arising from the formation of ductile plastic regions in the adjacent members. Joints must be designed in such a way that the required energy dissipation occurs in potential plastic hinges of adjacent members and not in the joint core region.

15.4.2 *Design forces*

15.4.2.1 *Forces acting on beam column joint*

The design forces acting on a beam column joint core shall be evaluated from the maximum internal forces generated by all the members meeting at the joint in equilibrium. The forces shall be those induced when the overstrengths of the beam or beams are developed, except in cases when a column is permitted to be the weaker member. Where a plastic hinge can develop in a beam adjacent to a joint, all tension reinforcement of the beam section, including that placed in flanges in accordance with 9.4.1.6, shall be taken into account. Where plastic hinges are to develop in columns rather than in beams, nominal joint shear strength shall be based on the overstrength of the columns.

Where detailing ensures that plastic regions are located away from the joint face, the design joint shear forces shall be calculated from the forces occurring at the joint face at the overstrength of the plastic regions.

15.4.2.2 *Horizontal design shear force*

The magnitude of the horizontal design shear force in the joint, V_{jh}^{*}, shall be evaluated from a rational analysis taking into account the effect of all forces acting on the joint.

15.4.2.3 *Consideration of concurrency*

At columns of two-way frames where beams frame into the joint from two directions, these forces need only be considered in each direction independently. However, axial column forces caused by beam plastic hinges in two directions should be considered.

15.4.3 *Design assumptions*

15.4.3.1 *The role of shear reinforcement*

The design of the shear reinforcement in the joint shall be based on the prevention of premature bond failure and effective control of a potential tension failure plane that extends from one corner of the joint to the diagonally opposite edge.

15.4.3.2 *Maximum horizontal design shear force*

The horizontal design shear force across a joint for seismic stress reversals shall not exceed the smaller of $0.2f_c'b_jh_c$, or $10\ b_jh_c$ where b_j is defined in 15.3.4.

15.4.3.3 *Determination of shear resistance of joint*

The shear strength of joints shall be assessed as follows:

(a) The shear resistance of beam column joints shall be based on a mechanism consisting of a single diagonal concrete strut and a truss mechanism with horizontal and vertical stirrups, hoops or bars adequately anchored at the boundaries of the joint capable of sustaining a diagonal concrete compression field;

(b) Other forms of joint shear reinforcement such as beam bars bent diagonally across the joint in one or both directions, or large diameter hoops placed outside the joint core where horizontal beam haunches allow this to be done, may also be used if it can be shown by rational analysis or tests or both that the required joint shear and anchorage forces can be adequately transferred.

15.4.3.4 *Horizontal joint shear reinforcement*

The requirements of 15.4.4 and 15.4.5 shall apply when plastic hinges can develop in the beams at the column face. Where plastic hinges in columns adjacent to a joint are permitted, application of 15.4.4 and 15.4.5 shall be correspondingly interchanged. Where plastic hinges can develop in the beams, but these are remote from the column face, the design joint shear forces shall be calculated in accordance with 15.4.2.1, and joint reinforcement and detailing shall be in accordance with 15.3.5 to 15.3.8. As forces are derived from overstrengths, a ϕ value of 1.0 shall be used in accordance with 2.3.2.2.

15.4.3.5 *Placement of shear reinforcement*

The required horizontal and vertical joint shear reinforcement shall be placed within the effective width of the joint, b_j; defined in 15.3.4, relevant to each direction of loading.

15.4.3.6 *Design yield strength of shear reinforcement*

The design yield strength of shear reinforcement, f_{yh} and f_{yv}, shall not exceed 500 MPa.

15.4.4 *Horizontal joint shear reinforcement*

15.4.4.1 *Area of horizontal joint shear reinforcement*

The area of total effective horizontal joint shear reinforcement corresponding to each direction of horizontal joint shear force shall be:

(a) *For interior joints*

A_{jh} shall be determined from Equation 15–9

$$A_{jh} = \frac{6V^{*}_{ojh}}{f'_c b_j h_c}\left(\frac{\alpha_i f_y A^{*}_s}{f_{yh}}\right) \qquad \text{(Eq. 15–9)}$$

where

$$0.85 \le \left[\frac{6V^{*}_{ojh}}{f'_c b_j h_c}\right] \le 1.20$$ A2

and

(i) $\alpha_i = 1.4\alpha_n$

or, where the beneficial effects of axial compression loads acting above the joint are included;

$$\alpha_i = \left(1.4 - 1.6\frac{C_j N^{*}_o}{f'_c A_g}\right)\alpha_n$$

where

$\alpha_n = 0.85$ where the sectional curvature ductility of the plastic region adjacent to the joint is equal to or less than that for LDPR (see 2.6.1.3.1)

$\alpha_n = 1.0$ where the sectional curvature ductility of the plastic region adjacent to the joint is equal to or less than that for DPR (see 2.6.1.3.1)

(ii) A_s^* is the greater of the area of top or bottom beam reinforcement passing through the joint. It excludes bars in effective tension flanges.

(b) *For exterior joints*

A_{jh} shall be determined from Equation 15–10

A2

$$A_{jh} = \frac{6V_{ojh}^*}{f_c' b_j h_c}\left(\frac{\beta f_y A_s}{f_{yh}}\right)\left(0.7 - \frac{C_j N_o^*}{f_c' A_g}\right) \quad \text{.....(Eq. 15–10)}$$

where

A2

$$0.85 \le \left[\frac{6V_{ojh}^*}{f_c' b_j h_c}\right] \le 1.20$$

and N_o^* is taken negative for axial tension in which case $C_j = 1$ must be assumed, and β = ratio of area of compression beam reinforcement to area of tension beam reinforcement, not to be taken larger than unity.

(c) The area A_{jh} to be provided in accordance with 15.4.4.1 (a) and (b) shall be equal to or greater than $0.4V_{ojh}^*/f_{yh}$.

15.4.4.2 *Prestressed beams*

Where beams are prestressed through the joint, the horizontal joint shear reinforcement required by 15.4.4.1 may be reduced by:

$$\Delta A_{jh} = \frac{0.7P_{cs}}{f_{yh}} \quad \text{.....(Eq. 15–11)}$$

where P_{cs} is the force after all losses in the prestressing steel that is located within the central third of the beam depth.

A1

15.4.4.3 *Distribution of horizontal joint shear reinforcement*

The effective horizontal joint shear reinforcement crossing the potential diagonal failure plane shall consist of sets of stirrups or hoops or intermediate ties or equivalent reinforcement placed between but not immediately adjacent to the innermost layers of the top and bottom beam reinforcement, and shall be distributed as uniformly as practicable. Any tie leg bent around column bars that does not cross the potential diagonal failure plane, shall be neglected.

A1

15.4.4.4 *Minimum horizontal joint reinforcement*

The quantity of horizontal joint reinforcement, placed as required by 15.4.4.3, shall be equal to or greater than that required by 10.4.7.4 and 10.4.7.5 for confinement of concrete and lateral restraint of bars in the end regions of columns immediately above or below a joint. The vertical spacing of sets of ties or hoops within a joint shall not exceed 10 times the diameter of the smallest column bar or 200 mm, whichever is less.

15.4.5 ***Vertical joint shear reinforcement***

A2

15.4.5.1 *Area of vertical joint reinforcement*

Where columns are designed so that primary plastic hinges do not form against the beam face of the joint zone, the total area of effective vertical joint shear reinforcement shall be determined for each of the two directions for interior and exterior joints from Equation 15–12.

$$A_{jv} = \alpha_v A_{jh} \frac{f_{yh}}{f_{yv}} \frac{h_b}{h_c} \quad \text{.....(Eq. 15–12)}$$

where

$$\alpha_v = \frac{0.7}{1 + \frac{N_o^*}{f_c' A_g}} \quad \text{.....(Eq. 15–13)}$$

Where a primary plastic hinge may form in the column against the beam face, the vertical joint zone reinforcement should be designed on the same basis as the horizontal joint shear reinforcement for a joint zone with beams, which may form plastic regions against the column face or faces. A2

15.4.5.2 *Vertical joint shear reinforcement*

The vertical joint shear reinforcement shall consist of intermediate column bars, placed in the plane of bending between corner bars, or vertical stirrup ties or special vertical bars, placed in the column and adequately anchored to transmit the required tensile forces within the joint. The total area of effective vertical joint shear reinforcement shall be placed within the effective joint area, $b_j h_c$, as defined by 15.3.4.

15.4.5.3 *Spacing of vertical joint reinforcement*

The horizontal spacing of vertical joint reinforcement in each plane of any beam framing into a joint shall not exceed the larger of one-quarter of the adjacent lateral dimension of the section or 200 mm, and in all cases there shall be at least one intermediate bar in each side of the column in that plane.

15.4.6 *Joints with wide columns and narrow beams*

Where the width of the column is larger than the effective joint width specified in 15.3.4 or 15.4.7, all flexural reinforcement in the column that is required to interact with the narrow beam shall be placed within the effective joint area, $b_j h_c$. Additional longitudinal column reinforcement shall be placed outside of this effective joint area in accordance with 10.4.6.2. Transverse reinforcement outside of the effective joint area shall be in accordance with the confinement provisions of 10.4.7.4 and 10.4.7.5.

15.4.7 *Eccentric beam column joints*

The effective joint width, b_j, shall not exceed $0.5(b_w + b_c + 0.5\,h_c) - e$, where e is the eccentricity of a beam relative to the column into which it frames and equals the distance between the centrelines of the webs of the beam and the column.

15.4.8 *Maximum diameter of longitudinal beam bars passing through joints*

The diameter of longitudinal beam bars passing through the beam column joint shall be in accordance with 9.4.3.5.

15.4.9 *Maximum diameter of column bars passing through joint*

The diameter of longitudinal column bars passing through the beam column joint shall be in accordance with 10.4.6.6.

NOTES

16 BEARING STRENGTH, BRACKETS AND CORBELS

16.1 Notation

a shear span, distance between concentrated load and face of support, mm

A_1 loaded area, mm^2

A_2 the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of one vertical to two horizontal, mm^2

A_f area of reinforcement in bracket or corbel resisting moment $[V^*a + N_c^*(h - d)]$, mm^2

A_h area of closed corbel stirrups or ties, mm^2

A_n area of reinforcement in bracket or corbel resisting tensile force N_c^*, mm^2

A_s area of non-prestressed tension reinforcement

A_{vf} area of shear-friction reinforcement, mm^2

b_w width of web, mm

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement at the springing of a corbel, mm

f_c' specified compressive strength of concrete, MPa

f_ℓ average confining stress at the perimeter of a loaded area, MPa

f_y lower characteristic yield strength of steel reinforcement, MPa

h overall depth of corbel, mm

N_c^* design tensile force applied at top of bracket or corbel acting simultaneously with V^* to be taken as positive if tension, N

p proportion of flexural reinforcement, $A_s/b_w d$

V^* design shear force, N

V_n nominal shear strength of section, N

ϕ strength reduction factor

16.2 Scope

The provisions of this section apply to bearing strength, brackets and corbels.

16.3 Bearing strength

16.3.1 *General*

Design bearing strength of concrete shall not exceed $\phi\,(0.85\,f_c'\,A_1)$, except when the supporting surface is wider on all sides than the loaded area, then the design bearing strength of the loaded area may be multiplied by $\sqrt{A_2 / A_1}$ but by not more than two.

16.3.2 *Exclusions*

The bearing pressure limit given in 16.3.1 may be exceeded where either:

(a) Extensive tests have shown the bearing pressure can be sustained without any reduction in safety index.

(b) The loaded area is confined, by reinforcement or some other means, by a confining stress f_ℓ, such that the bearing pressure is equal to or less than:

$$\phi(0.85f_c' + 4f_\ell) \quad \text{(Eq. 16–1)}$$

where ϕ is the strength reduction factor and f_ℓ may be taken as the average confining stress at the perimeter of the loaded area, but f_ℓ shall not exceed 1.5 times the minimum confining pressure.

16.3.3 *Strength reduction factor*

The strength reduction factor shall be taken as 0.65, except it may be taken as 0.85 where the concrete is confined by reinforcement such that the confining pressure, f_ℓ, is equal to or greater than 0.08 times the maximum bearing pressure.

16.4 Design of brackets and corbels

16.4.1 *Strength reduction factor*

In all design calculations in accordance with 16.4, the strength reduction factor ϕ shall be taken equal to 0.75.

16.4.2 *Loading*

The corbel or bracket shall be designed to sustain simultaneously the design vertical force, V^* and a horizontal force N_c^*, which acts at the same position as V^* and applies tension to the member. Unless special precautions are taken to avoid the development of N_c^*, it shall be regarded as a live load with a magnitude equal to or greater than 0.2 V^*.

16.4.3 *Bearing area*

The bearing area for load on a bracket or corbel shall not project beyond the straight portion of the primary tension bars, A_s, nor project beyond the interior face of a transverse anchor bar (if one is provided).

16.4.4 *Method of design*

Brackets and corbels shall be designed by either method (a), or method (b) where appropriate:

(a) Brackets or corbels with a span to effective depth ratio (a/d) of 1.8 or less may be designed by the strut and tie method.

(b) Brackets or corbels with a span to effective depth ratio (a/d) of 1.0 or less may be designed by the empirical approach given in 16.5.

16.5 Empirical design of corbels or brackets

16.5.1 *Depth at outside edge*

The depth at the outside edge of the bearing area shall be equal to or greater than 0.5d.

16.5.2 *Design actions at face of support*

The section at the face of a support shall be designed to resist simultaneously a shear V^*, a moment $[V^*a + N_c^*(h - d)]$, and a horizontal tensile force N_c^*, acting at the level of the flexural tension reinforcement closest to the tension side of the corbel.

16.5.3 *Shear-friction reinforcement*

Design of shear-friction reinforcement A_{vf} to resist shear V^* shall be in accordance with 7.7.

16.5.4 *Maximum shear stress*

The nominal shear strength at the support face for normal weight concrete shall be the smaller of 0.2 $f_c' b_w d$ or $8 b_w d$. For all-lightweight or sand-lightweight concrete, shear strength V_n shall be equal to or less than the smaller of (0.2 - 0.07 a/d) $f_c' b_w d$ and $(8.0 - 1.9a/d) b_w d$.

16.5.5 *Reinforcement for flexure*

Reinforcement A_f to resist moment $V^*a + N_c^*(h - d)$ shall be computed in accordance with 7.4.

16.5.6 *Reinforcement for axial tension force*

Reinforcement A_n to resist tensile force N_c^* shall be determined from $N_c^* < \phi A_n f_y$.

16.5.7 *Primary tension reinforcement*

The area of primary tension reinforcement A_s shall be made equal to the greater of $(A_f + A_n)$ or $(2A_{vf}/3 + A_n)$.

16.5.8 *Closed stirrups or ties*

Closed stirrups or ties parallel to A_s, with a total area A_h equal to, or greater than $0.5(A_s - A_n)$, shall be uniformly distributed within two-thirds of the effective depth adjacent to A_s.

16.5.9 *Minimum ratio for p*

Ratio $p = A_s/b_w d$ shall be equal to or greater than $0.04\ (f'_c/f_y)$.

16.5.10 *Reinforcement A_s*

At the front face of bracket or corbel, primary tension reinforcement A_s shall be anchored by one of the following:

(a) By a structural weld to a transverse bar of at least equal size; the weld is to be designed to develop lower characteristic yield strength f_y of A_s bars;

(b) By bending primary tension bars A_s back to form a horizontal loop; or

(c) By some other means of positive anchorage.

NOTES

17 EMBEDDED ITEMS, FIXINGS AND SECONDARY STRUCTURAL ELEMENTS

17.1 Notation

A_{brg} bearing area of the head of stud or anchor, mm^2

A_n projected area of the assumed 35° failure surface, mm^2

A_{no} projected concrete failure area of one anchor when not limited by edge distance, mm^2

A_{se} effective cross-sectional area of an anchor, mm^2

A_v projected concrete failure area of an anchor or group of anchors in shear, mm^2

A_{vo} projected concrete failure area of an anchor or group of anchors in shear, when not limited by corner influences, spacing, or member thickness, mm^2

c distance from the centre of an anchor shaft to the edge of the concrete, mm

c_1 distance from the centre of an anchor resistance of an anchor to the edge of the concrete in the direction in which the load is applied, mm

c_2 distance from centre of shaft to the edge of the concrete, perpendicular to c, mm

c_{max} largest edge distance, mm

c_{min} smallest edge distance, mm

d_o outside diameter or shaft diameter of the anchor. Shall be taken as 0.8 times the width of a hooked steel plate, mm

e_n distance from the inner surface to the outer tip of a hooked bolt, mm

e'_n the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension (always taken as positive), mm

e'_v distance between the point of shear force application and the centroid of the group of anchors resisting the shear in the direction of the applied shear, mm

f'_c specified compressive strength of concrete, MPa

f_{ut} tensile strength of the anchor, but shall not be taken greater than $1.9f_y$, MPa

f_y specified yield strength of anchor steel, MPa

h_{ef} effective anchor embedment depth, mm

ℓ load-bearing length of anchors for shear, equal to h_{ef} for anchors with constant stiffness over the full length of the embedded section but less than $8d_o$. Shall be taken as 0.8 times the effective embedment depth for hooked metal plates, mm

k coefficient for basic concrete breakout strength in tension

k_{cp} coefficient of pry-out strength

k_1 multiplier for edge distance

k_2 0.6 for cast-in headed studs, headed bolts, or hooked bolts, or hooked steel plates

n number of anchors in a group

N_b basic concrete breakout strength in tension of a single anchor in cracked concrete, N

N_{cb} nominal concrete breakout strength in tension of a single anchor, N

N_{cbg} nominal concrete breakout strength in tension of a group of anchors, N

N_n lower characteristic strength in tension, N

N_p pullout strength in tension of a single anchor in cracked concrete, N

N_{pn} lower characteristic pullout strength in tension of a single anchor, N

N_s nominal strength of a single anchor or group of anchors in tension as governed by the steel strength, N

N_{sb} side blowout strength of a single anchor, N

N^* design tension, N

s centre-to-centre spacing of the anchors, mm

ϕ strength reduction factor

V_b basic concrete breakout strength for a single anchor in shear, N

V_{cb} lower characteristic concrete breakout strength in shear of a single or group of anchors, N

V_{cp} lower characteristic concrete pry-out strength, N

V_n lower characteristic shear strength, N

V_s nominal strength in shear of a single anchor or group of anchors as governed by the steel strength, N

V^* design shear force, N

Ψ_1 modification factor, for strength in tension, to account for anchor groups loaded eccentrically

Ψ_2 modification factor, for strength in tension, to account for edge distances smaller than 1.5 h_{ef}

Ψ_3 modification factor, for strength in tension, to account for cracking

Ψ_4 modification factor for pullout strength, to account for cracking

Ψ_5 modification factor, for strength in shear, to account for anchor groups loaded eccentrically

Ψ_6 modification factor, for strength in shear, to account for edge distances smaller than $1.5c_1$

Ψ_7 modification factor, for strength in shear, to account for cracking

17.2 Scope

The requirements of this section apply to conduits or pipes embedded within structural concrete and to fixings and connections that are likely to transmit forces to a concrete structure or between elements of a structure.

17.3 Design procedures

Elements within the scope of this section shall be designed in accordance with the procedures of 2.2, 2.3 and 2.4 as appropriate.

17.4 Embedded items

Conduits or pipes shall not significantly impair the strength of the construction.

17.5 Fixings

17.5.1 *General*

Fixings, including holding-down bolts, inserts and ferrules and associated hardware, shall comply with 17.5.2 to 17.5.9. Fixings subjected to seismic actions shall satisfy the requirements of 17.6.

17.5.2 *Design forces*

A fixing shall be designed to transmit all the actions set out in AS/NZS 1170 or other referenced loading standard for the ultimate limit state. The design actions shall also include forces induced in the connection due to creep, shrinkage, temperature effects and relative deformation between the attached items.

17.5.3 *Inserts for lifting*

Design of inserts for lifting shall be in accordance with the Approved Code of Practice for the Safe Handling, Transportation and Erection of Precast Concrete published by the Department of Labour.

17.5.4 *Strength of fixings by testing*

A2 | The nominal strength of fixings may be based upon tests to evaluate the 5 percentile fracture, or by calculation, of the following:

(a) Steel strength of fixing in tension
(b) Steel strength of fixing in shear
(c) Concrete breakout strength of fixing in tension
(d) Concrete breakout strength of anchor in shear
(e) Pullout strength of anchor in tension
(f) Concrete side-face blow-out strength of anchor in tension
(g) Concrete pry-out strength of anchor in shear.

17.5.5 *Strength of fixings by calculation*

For anchors with diameters less than 50 mm, and embedded lengths less than 635 mm, calculations for the capacity of cast-in-place mechanical fasteners without supplementary reinforcement in cracked and uncracked concrete shall be determined in accordance with either 17.5.6, or ACI 318 Appendix D. The effect of supplementary reinforcement on the restraint of concrete breakout may be included by rational analysis.

Clause 17.5.6 or ACI 318 Appendix D may also be used for post-installed mechanical anchors that have passed the qualification test stipulated in ACI 355.2. Post-installed mechanical anchors intended to resist seismic actions shall have passed the simulated seismic test of ACI 355.2.

17.5.6 *Strength of cast-in anchors*

17.5.6.1 *Scope*

The design method outline in this section applies to cast-in anchors, without supplementary reinforcement. Speciality inserts, trough bolts, multiple anchors connected to a single plate at the embedded end of the anchors, adhesive or grouted anchors, and direct anchors such as powder or pneumatic actuated nails or bolts are not included.

17.5.6.2 *Load application*

Load application involving high cycle fatigue or impact loads are also not covered by this section.

17.5.6.3 *Strength requirements*

In the design of anchors

$N^* \leq \phi N_n$ .....(Eq. 17–1)

and

$V^* \leq \phi V_n$ .....(Eq. 17–2)

where N_n is the lower characteristic strength in tension and is given by 17.5.7 and V_n is the lower characteristic shear strength given by 17.5.8.

17.5.6.4 *Strength reduction factors*

The strength reduction factors shall be:

(a) Shear: $\phi = 0.75$.....(Eq. 17–3)

(b) Tension: $\phi = 0.65$.....(Eq. 17–4)

17.5.6.5 *Interaction of tension and shear*

Resistance to combined tensile and shear actions shall be considered in design using interaction expressions that result in computation of strength in substantial agreement with results of comprehensive test. This requirement shall be considered satisfied by 17.5.6.6.

17.5.6.6 *Interaction of tension and shear – simplified procedures*

Unless determined in accordance with 17.5.6.5, anchors or groups of anchors shall be designed to satisfy the following:

(a) Where $V^* \leq 0.2\ \phi V_n$ then full strength in tension is permitted ($\phi N_n \geq N^*$)

(b) Where $N^* \leq 0.2\ \phi N_n$ then full strength in shear is permitted ($\phi V_n \geq V^*$)

(c) Where $V^* > 0.2\ \phi V_n$ and $N^* > 0.2\ \phi N_n$ then:

$$\frac{N^*}{\phi N_n} + \frac{V^*}{\phi V_n} \le 1.2 \quad \text{(Eq. 17–5)}$$

17.5.7 *Lower characteristic strength of anchor in tension*

The calculated lower characteristic strength of an anchor in tension, N_n, shall be the smaller of the following:

(a) The lower characteristic tensile strength of the steel of the anchor, N_s, 17.5.7.1;

(b) The lower characteristic concrete breakout strength of the anchor in tension, N_{cb}, 17.5.7.2;

(c) The lower characteristic pullout strength of the anchor in tension, N_{pn}, 17.5.7.3;

(d) The lower characteristic concrete side face blowout strength of the anchor in tension, N_{sb}, 17.5.7.4.

17.5.7.1 *Steel strength of anchor in tension*

The lower characteristic tensile strength of an anchor as governed by the steel, N_s, shall be given by:

$$N_s = nA_{se}f_{ut} \quad \text{(Eq. 17-6)}$$

where

n = number of anchors in a group,

A_{se} = effective cross-sectional area of an anchor, mm^2

f_{ut} = tensile strength of the anchor, but shall not be taken greater than $1.9f_y$, MPa.

17.5.7.2 *Strength of concrete breakout of anchor*

The lower characteristic breakout strength of an anchor, or group of anchors, in tension, N_{cb}, without supplementary reinforcement and normal weight concrete is given by:

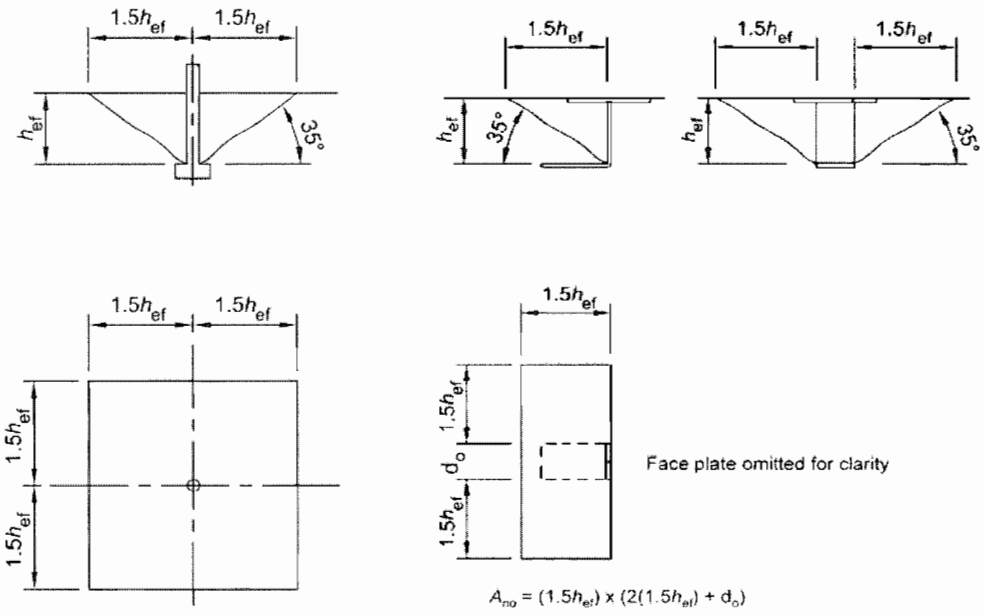
$$N_{cb} = \Psi_1\Psi_2\Psi_3 \frac{A_n}{A_{no}} N_b \quad \text{(Eq. 17–7)}$$

where

A_n = projected area of the assumed 35° failure surface taken from the outside of the head of the anchor or group of anchors (must be taken as less than nA_{no}). Where the perimeter of the group of anchors is closer than $1.5h_{ef}$ to any edge, consideration shall be given to the overlap of failure surfaces with the edge and corners of concrete panels, mm^2

A_{no} = projected concrete failure area of one anchor when not limited by edge distance, as shown in Figure 17.1 , mm^2.

A2



(a) Headed anchor (b) Hooked plate anchor

Figure 17.1 – Typical failure surface areas of individual anchors, not limited by edge distances

Ψ_1 = modification for anchor groups comprising of more than one anchor. Equal to 1.0 for a single anchor and where $e_n' < s/2$ it is given by:

$$\Psi_1 = \frac{1}{\left(1+\frac{2e_n'}{3h_{ef}}\right)} \leq 1.0 \quad \text{.....(Eq. 17–8)}$$

where

e_n' = the distance between the resultant tension load on a group of anchors in tension and the centroid of the group of anchors loaded in tension (always taken as positive), mm

h_{ef} = effective anchor embedment, mm

s = centre-to-centre spacing of the anchors, mm

Ψ_2 = modification factor for edge distances, given by:

(a) $\Psi_2 = 1.0$ when $c_{min} \geq 1.5h_{ef}$

or,

(b) $\Psi_2 = 0.7 + 0.3\,\frac{c_{min}}{1.5h_{ef}}$ when $c_{min} < 1.5h_{ef}$

Ψ_3 = modification for cracking of concrete, equals:

(a) $\Psi_3 = 1.25$ for cast-in anchors in uncracked concrete,

(b) $\Psi_3 = 1.0$ for concrete which is cracked at service load levels. Cracking in the concrete shall be controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5.

N_b = basic concrete breakout strength in tension of a single anchor in concrete cracked at service load levels but with the extent of cracking controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5, given by:

$$N_b = k\sqrt{f_c'}h_{ef}^{1.5} \quad \text{(Eq. 17–9)}$$

where

f_c' shall not be taken greater than 70 MPa

k = 10 for cast-in anchors

h_{ef} = effective anchor embedment depth, mm, however if three or more edges are closer than 1.5 h_{ef} to the anchor, h_{ef} shall be replaced by $c_{max}/1.5$ in Equation 17–9, where c_{max} is the largest edge distance of the influencing edges.

17.5.7.3 *Lower characteristic tension pullout strength of anchor*

The lower characteristic pullout strength of an anchor or group of anchors in tension shall be given by:

$$N_{pn} = \Psi_4 N_p \quad \text{(Eq. 17-10)}$$

where

N_{pn} = lower characteristic pullout strength in tension of a single anchor, N

Ψ_4 = modification factor for pullout strength

N_p = pullout strength of a single anchor in cracked concrete, N, given by:

(a) For a headed stud or headed bolt:

$$N_p = 8f_c'A_{brg} \quad \text{(Eq. 17–11)}$$

(b) For a hooked bolt where $3d_o \le e_h \le 4.5d_o$:

$$N_p = 0.9\, f_c'\, e_h d_o \quad \text{(Eq. 17–12)}$$

where

A_{brg} = bearing area of the head of stud or anchor, mm^2

d_o = outside diameter of anchor or shaft diameter of a headed stud, headed bolt or hooked bolt, mm

e_h = distance from the inner surface to the outer tip of a hooked bolt, mm

Ψ_4 = 1.0 for concrete cracked at service load levels but with the extent of cracking controlled by reinforcement distributed in accordance with 2.4.4.4 and 2.4.4.5

Ψ_4 = 1.4 for concrete with no cracking at service load levels.

17.5.7.4 *Lower characteristic concrete side face blowout strength*

The side face blowout strength of a headed anchor with deep embedment close to an edge ($c < 0.4h_{ef}$) shall not exceed:

$$N_{sb} = 13.3k_1 c\sqrt{A_{brg}f_c'} \quad \text{(Eq. 17–13)}$$

where

N_{sb} = side blowout strength of a single anchor, N

c = distance from the centre of an anchor shaft to the edge of the concrete, mm

k_1 = multiplier for edge distance, given by:

(a) When $c_2 \ge 3c$, k_1=1.0

(b) When $c < c_2 < 3c$, $k_1 = \dfrac{1+\frac{c_2}{c}}{4}$

where

c_2 = distance from centre of shaft to the edge of the concrete, perpendicular to c, mm.

17.5.8 *Lower characteristic strength of anchor in shear*

The calculated lower characteristic strength of an anchor in shear, V_n, shall be the smallest of the following;

(a) The lower characteristic shear strength of the steel of the anchor, V_s, 17.5.8.1;
(b) The lower characteristic concrete breakout strength of the anchor in shear, V_{cb}, 17.5.8.2, or 17.5.8.3;
(c) The lower characteristic concrete pry-out strength of the anchor in shear, V_{cp}, 17.5.8.4.

17.5.8.1 *Lower characteristic shear strength of steel of anchor*

The lower characteristic strength of an anchor, or group of anchors, in shear governed by the steel shall not exceed:

(a) For cast-in headed stud anchors:

$$V_s = n\,A_{se}f_{ut} \quad \text{.....(Eq. 17–14)}$$

(b) For cast-in headed bolts and hooked bolt anchors:

$$V_s = n\,0.6A_{se}f_{ut} \quad \text{.....(Eq. 17–15)}$$

where f_{ut} shall be less than $1.9f_y$ or 860 MPa.

17.5.8.2 *Lower characteristic concrete breakout strength of the anchor in shear perpendicular to edge*

The concrete breakout strength of an anchor, or group of anchors, in shear when loaded perpendicular to an edge shall not exceed:

$$V_{cb} = \frac{A_v}{A_{vo}}\Psi_5\Psi_6\Psi_7V_b \quad \text{.....(Eq. 17–16)}$$

For anchors or anchor groups located at or near corners the shear strength shall be determined in each direction.

where

V_{cb} = lower characteristic concrete breakout strength in shear of a single or group of anchors, N.
A_v = projected concrete failure area of an anchor or group of anchors in shear, mm^2 as shown in Figure 17.2(b) A2
A_{vo} = projected concrete failure area of an anchor in shear, when not limited by corner influences, spacing, or member thickness, mm^2 as shown in Figure 17.2(a)
V_b = basic concrete breakout strength for a single anchor in shear, N, for anchors at centre-to-centre spacing greater than 65 mm, is given by:

$$V_b = k_2\left(\frac{\ell}{d_o}\right)^{0.2}\sqrt{d_o f_c'}\,(c_1)^{1.5} \quad \text{.....(Eq. 17–17)}$$

where

k_2 = 0.6 for cast-in headed studs, headed bolts, or hooked bolts, or hooked steel plates.

ℓ = load-bearing length of anchors for shear, equal to h_{ef} for anchors with constant stiffness over the full length of the embedded section but less than $8d_o$. Shall be taken as 0.8 times the effective embedment depth for hooked metal plates.

d_o = outside diameter or shaft diameter of the anchor. Shall be taken as 0.8 times the width of a hooked steel plate, mm.

c_1 = distance from the centre of resistance of an anchor to the edge of the concrete in the direction which the load is applied, mm. For hooked bars or hooked plates, c_1 is the lesser of the distance from either the centre of the anchor or the centre of the bend radius to the nearest edge in the direction of the applied shear force. For straight anchors it shall be taken to the centre of the shaft of the anchor. For the special case for anchors influenced by three or more edges, c_1 shall be limited to $h/1.5$.

A2

Ψ_5 = modification factor for anchor groups, where $e_v' < \frac{s}{2}$ is given by:

$$\Psi_5 = \frac{1}{1+\frac{2e_v'}{3c_1}} \leq 1.0 \qquad \text{(Eq. 17–18)}$$

where

e_v' = the distance between the point of shear force application and the centroid of the group of anchors resisting the shear in the direction of the applied shear, mm

Ψ_6 = modification factor for edge distance given by:

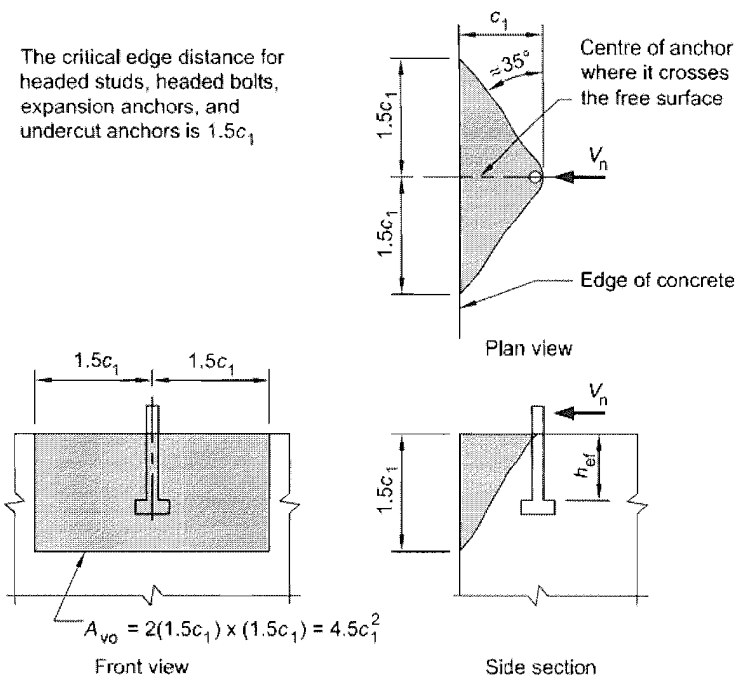
(a) For $c_2 \geq 1.5c_1$ $\quad \Psi_6 = 1.0$ (Eq. 17–19)

or

(b) For $c_2 < 1.5c_1$ $\quad \Psi_6 = 0.7 + 0.3\frac{c_2}{1.5c_1}$ (Eq. 17–20)

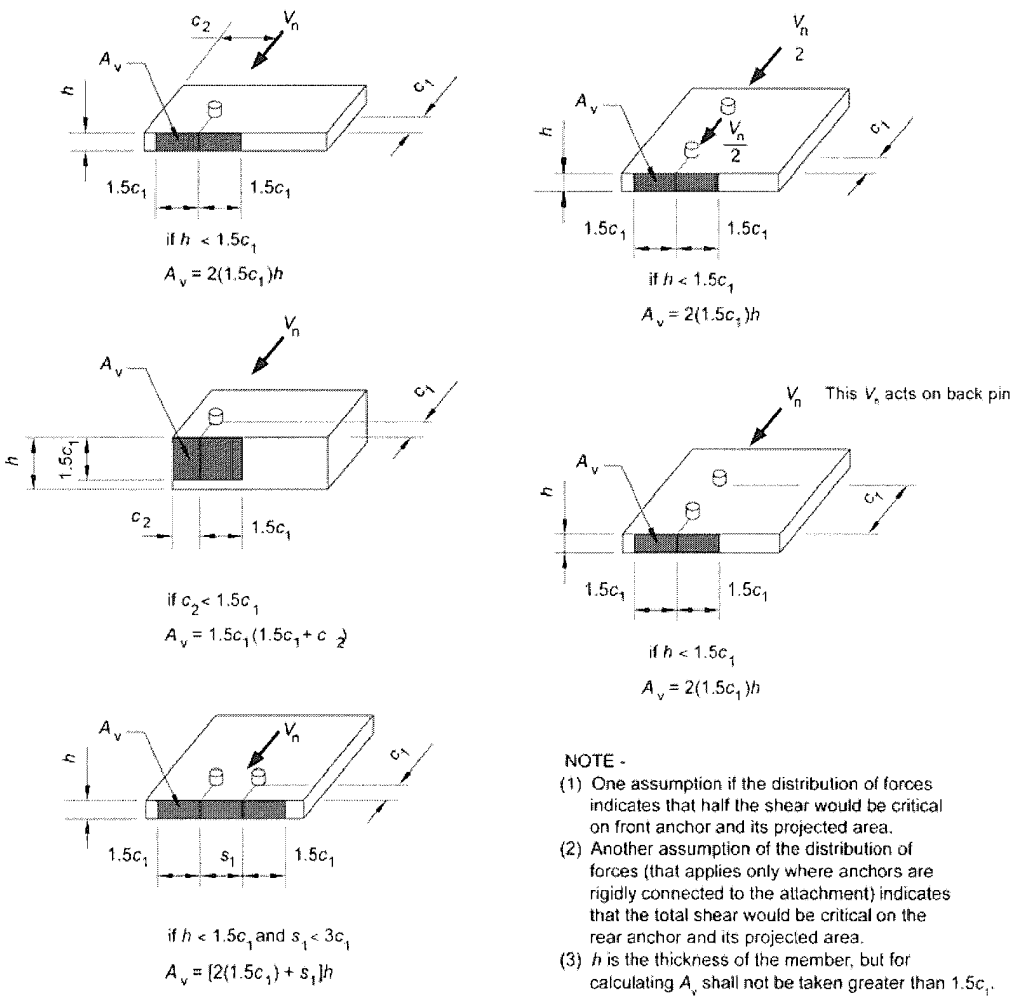
Ψ_7 = modification factor for cracked concrete, given by:

- Ψ_7 = 1.0 for anchors in cracked concrete with no supplementary reinforcement
- Ψ_7 = 1.2 for anchors in cracked concrete with supplementary minimum of a 12 mm diameter reinforcing bar as supplementary reinforcement.
- Ψ_7 = 1.4 for concrete which is not cracked at service load levels.



(a) Calculation of A_{vo}

A2



(b) Projected area for single anchors and groups of anchors and calculation of A_v

Figure 17.2 – Determination of A_v and A_{vo} for anchors

17.5.8.3 *Lower characteristic concrete breakout strength of the anchor in shear parallel to edge*

The concrete breakout strength of an anchor or group of anchors, in shear when loaded parallel to an edge shall not exceed:

$$V_{cb} = 2\frac{A_v}{A_{vo}}\Psi_5\Psi_7 V_b \quad \text{.....(Eq. 17–21)}$$

A2 | where A_v, A_{vo}, Ψ_5, Ψ_7 and V_b are defined in 17.5.8.2.

17.5.8.4 *Lower characteristic concrete pry-out of the anchor in shear*

The nominal pry-out strength of an anchor shall not exceed:

$$V_{cp} = k_{cp}N_{cb} \quad \text{.....(Eq. 17–22]}$$

where

V_{cp} = lower characteristic concrete pry-out strength, N

N_{cb} = nominal concrete breakout strength in tension of a single anchor, N

k_{cp} = coefficient of pry-out strength, given by:

(a) For $h_{ef} < 65$ mm, $k_{cp} = 1.0$;

(b) For $h_{ef} \geq 65$ mm, $k_{cp} = 2.0$.

17.5.9 ***Durability and fire resistance***

The cover for durability and fire resistance shall be in accordance with Sections 3 and 4 respectively.

17.6 Additional design requirements for fixings designed for earthquake effects

17.6.1 ***Fixing design philosophy***

A2 | Fixings shall be designed to prevent failure in an earthquake. The design philosophy adopted to achieve this shall be one of (a) to (d), or a combination of these:

(a) Fixings shall be designed to accommodate relative seismic movement by separation (17.6.2);

(b) The strengths of the fixings are greater than the actions associated with ductile yielding of the attachment (17.6.3);

(c) Fixings shall be designed to remain elastic (17.6.4);

(d) The fixing is designed to accommodate the expected seismic actions and deformations in a ductile manner (17.6.5).

17.6.2 ***Fixings designed for seismic separation***

When seismic deflection of the structure results in relative movement between an element and the points on the structure to which it is fixed, the fixings shall be designed to give clearance for the relative movements at these fixing points, corresponding to 1.5 times the seismic deflection at the ultimate limit

A2 | state computed from NZS 1170.5. Frictional forces that may be present on sliding surfaces shall be allowed for in the design of the fixing.

17.6.3 ***Fixings stronger than the overstrength capacity of the attachment***

When this design philosophy is adopted, the components of the fixing shall be designed so that the capacity of the fixing exceeds the development of overstrength yielding of the attachment. The design actions on the fixings shall be determined assuming overstrength actions determined in accordance with 2.6.5.4 and shall include consideration of plastic hinge elongation, creep, shrinkage and temperature effects. The capacity of the fixings shall be determined in accordance with 17.5.4 or 17.5.5 using the strength reduction factors of 17.5.6.4.

17.6.4 ***Fixings design to remain elastic***

Fixings designed to remain elastic shall be designed for the actions described in Clause 8.7 of NZS 1170.5. The capacity of the fixings shall be determined in accordance with 17.5.4 or 17.5.5 using 0.75 times the strength reduction factors of 17.5.6.4.

17.6.5 *Fixings designed for ductility*

Fixings may be designed for ductility for the actions described in Clause 8.7 of NZS 1170.5, when the actions and relative movement do not require deformations in the fixings in excess of twice their yield deformations. The calculation of the deformation in the connection shall include consideration of inter-storey drift, plastic hinge elongation, creep, shrinkage and temperature effects. The connection shall be designed to prevent non-ductile failure modes.

17.6.6 *Fixings in plastic hinge regions*

In regions of potential plastic hinging, the contribution of the cover concrete to the anchorage of fixings shall be ignored.

NOTES

18 PRECAST CONCRETE AND COMPOSITE CONCRETE FLEXURAL MEMBERS

18.1 Notation

A_g gross area of section. For hollow section, A_g is the area of concrete only and does not include the area of voids, mm^2

b_v width of cross section or effective width of interface in a section between precast and cast *in situ* concrete (see 18.5.4.2), mm A2

d distance from extreme compression fibre to centroid of tension reinforcement, mm4

f_y lower characteristic yield strength of non-prestressed reinforcement, MPa

h overall depth of member, mm

I moment of inertia of composite section, mm^4

Q first moment of area beyond the shear plane, being considered about the axis of bending, mm^3

$v_{d\ell}$ nominal longitudinal shear stress at any cross section or the nominal shear stress on the interface between the precast concrete shell and the cast-in-place core of the beam, MPa

v_ℓ maximum permissible longitudinal shear stress, MPa A2

V_L longitudinal shear force, N

V_n nominal shear strength of section, N

ϕ strength reduction factor, see 2.3.2.2 A2

18.2 Scope

18.2.1 *Precast concrete defined*

Provisions of this section apply for design of precast concrete members or structures, defined as those with structural elements that have been cast at a location other than in their final position in the structure.

18.2.2 *Composite concrete flexural members defined*

Also covered in this section are composite concrete flexural members defined as precast or cast-in-place concrete flexural members constructed in separate placements but interconnected so that all elements respond to loads and forces as a unit.

18.2.3 *Composite concrete and structural steel not covered*

Composite compression members of mixed concrete and structural steel sections shall be designed in accordance with 10.3.11. Other composite members of mixed concrete and structural steel shall be designed in accordance with NZS 3404.

18.2.4 *Section 18 in addition to other provisions of this Standard*

All provisions of this Standard, not specifically excluded and not in conflict with the provisions of section 18 shall also apply to structures incorporating precast and composite concrete structural members.

18.3 General

18.3.1 *Design to consider all loading and restraint conditions*

The design of precast members and connections shall consider all loading and restraint conditions from initial fabrication to completion of the structure, including those resulting from removal from the mould, storage, transportation, erection and propping. Design for temporary load cases during construction shall take account of the actual concrete strength at the relevant ages or stages of construction.

18.3.2 *Include forces and deformations at connections*

When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

18.3.3 *Consider serviceability and ultimate limit states*

Deflections and end rotations at the serviceability limit state shall be considered in addition to ultimate limit state actions.

18.3.4 *Tolerances*

Tolerances for dimensions of members and for their locations in the structure shall be specified by the designer. Design of precast members, connections and supports, shall include the effects of these tolerances. Combinations of secondary effects and construction tolerances shall be considered in designing bearing and/or hanger supports for precast concrete members.

18.3.5 *Long-term effects*

Long-term creep, shrinkage, temperature, differential settlement of foundations and restraint conditions shall be considered in the design and detailing of precast concrete members and their supports and connections.

18.4 Distribution of forces among members

18.4.1 *Forces perpendicular to plane of members*

Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

18.4.2 *In-plane forces*

Where the system behaviour requires in-plane forces to be transferred between the members of a precast floor or wall system, then the following shall apply:

(a) In-plane force paths shall be continuous through both connections and members; and

(b) Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

18.5 Member design

18.5.1 *Prestressed slabs and wall panels*

In one-way prestressed floor and roof slabs and in one-way, prestressed wall panels, all not wider than 2.4 m and where members are not mechanically connected so as to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of 8.8 for the precast unit in the direction normal to the flexural reinforcement may be waived. This waiver shall not apply to members that require reinforcement to resist transverse flexural stresses or to untopped precast floor units. In the context of this clause "prestressed" is defined as having equal to or greater than 1.5 MPa average residual concrete compression.

18.5.2 *Composite concrete flexural members*

18.5.2.1 *Shored and unshored members*

No distinction shall be made between shored and unshored members in the design for flexural strength of composite members for the ultimate limit state.

18.5.2.2 *Design of constituent elements*

Constituent elements shall be designed to support all loads that may be introduced prior to full development of the design strength of composite members.

18.5.2.3 *Reinforcement for composite members*

Reinforcement shall be provided as required for strength; and to control cracking and to prevent separation or slippage of individual elements of composite members.

18.5.3 *Shear resisted composite and non-composite section*

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Concrete elements prior to being made composite, and as composite members, shall be designed for the shear forces they may sustain at the ultimate limit state.

18.5.4 *Longitudinal shear in composite members*

18.5.4.1 *Requirements for full shear transfer*

In a composite member, transfer of longitudinal shear forces to the limits specified in 18.5.4.3 shall be assured at contact surfaces of interconnected elements when:

(a) Contact surfaces are clean, free of laitance, and roughened with a peak to trough amplitude equal to or greater than 5 mm; and/or

(b) Minimum ties are provided in accordance with 18.5.5; and/or

(c) Precast floor units are produced by a dry concrete mix extrusion process followed by surface treatment that leaves the top surface with a peak to trough roughness equal to or greater than 2 mm, clean and free of laitance, and provides shear keys equal to or greater than 20 mm wide at not greater than 1200 mm centres are used in the composite construction.

(d) Where the contact interface between in-situ and precast concrete is subject to specialised processes to ensure complete bonding by curing regimes or chemical processes such as wet to dry epoxy the above clauses (a) – (c). need not apply. The validity of such bonding processes shall be proved by cross joint shear tests.

If the requirements of (a) or (b) are not satisfied, longitudinal shear shall be investigated in accordance with 7.5.2 or 18.5.4.2.

18.5.4.2 *Longitudinal shear stress* A2

The longitudinal shear may be evaluated by computing the actual compressive or tensile force in any segment, with provisions made to transfer that force as longitudinal shear to the reacting element. Shear stress so derived shall not exceed values given by 18.5.4.3.

The longitudinal shear stress, $v_{d\ell}$, may be calculated at any cross section, except in potential plastic regions, by: A2

(a) For uncracked concrete sections, or sections where concrete in tension is neglected in calculating the section properties:

$$v_{d\ell} = \frac{V^* Q}{\phi I b_v} \quad \text{.....(Eq. 18–1)}$$

where b_v is the width of the section at the level being investigated, while in shell beams it may be taken as the width of the interface between the *in situ* concrete and the shell at level being investigated, plus twice the width of the upstanding sides of the shell. A2

(b) For cracked reinforced concrete with zero axial load

$$v_{d\ell} = \frac{V^*}{\phi j d b_v} \quad \text{.....(Eq. 18–2)}$$

where jd is the internal level arm of the flexural forces in the section and ϕ is 0.75. A2

18.5.4.3 *Transfer of longitudinal shear at contact surfaces*

The shear stress may be transferred at contact surfaces using the maximum nominal longitudinal shear stress, v_ℓ, as follows: A2

(a) Where ties are not provided, but the contact surfaces are clean and roughened or produced by the dry mix extrusion process referred to in 18.5.4.1(c), v_ℓ shall be equal to or smaller than 0.55 MPa; A2

(b) Where the minimum tie requirements of 18.5.5 are provided and the contact surfaces are clean but not roughened, v_ℓ shall be equal to or smaller than 0.55 MPa; A2

(c) Where the minimum tie requirements of 18.5.5 are satisfied and the contact surfaces are clean and adequately roughened in accordance with 18.5.4.1(a), v_ℓ shall be equal to or smaller than 2.4 MPa; A2

(d) Where $v_{d\ell}$ exceeds 2.4 MPa, design for longitudinal shear shall be carried out in accordance with 7.7.

18.5.4.4 *Transfer of shear where tension exists*

Where tension exists perpendicular to any surface, shear transfer by contact shall be assumed only when the minimum tie requirements of 18.5.5 are satisfied.

18.5.4.5 *Requirements for bridge superstructures*

For the main flexural members of bridge superstructures, ties equal or in excess of that required by 18.5.5 shall always be provided. Contact surfaces shall always be adequately roughened.

18.5.4.6 *Bridge deck overlays*

For the rehabilitation of an existing bridge deck through the application of an overlay, as an alternative to the requirements of 18.5.4.5, the design approach outlined in C18.5.4.6 of the Commentary may be adopted.

18.5.5 *Ties for longitudinal shear*

18.5.5.1 *Minimum anchorage into composite topping*

Adequately extended and anchored shear reinforcement may be included as contributing toward the resistance of longitudinal shear. The minimum thicknesses of normal density, composite topping concrete with compression strength of at least 25 MPa that stirrups, ties or spirals with 20 mm cover may effectively be anchored in are:

6 mm stirrups, ties or spirals.....50 mm minimum topping
10 mm stirrups, ties or spirals.....75 mm minimum topping
12 mm stirrups, ties or spirals.....90 mm minimum topping
16 mm stirrups, ties or spirals.....105 mm minimum topping

If cover greater than 20 mm is required, the thickness of topping indicated above shall be increased by the amount of additional cover.

18.5.5.2 *Minimum area and spacing of ties*

Where transverse bars or stirrups, ties or spirals are used to transfer longitudinal shear, the tie area shall be equal to or greater than that required by 9.3.9.4.15 and the spacing shall not exceed four times the least dimension of the supported element or 600 mm.

18.5.5.3 *Types of ties*

Ties for longitudinal shear may consist of single bars, multiple leg stirrups, spirals, headed studs, or the vertical legs of welded wire fabric. All ties shall be fully anchored into the components in accordance with 7.5.7.

18.5.6 *Precast shell beam construction*

18.5.6.1 *Section and material properties*

The designer shall take into account the various actions of the section as a whole and whether fully composite behaviour or that of only the cast-in-place core of the beam is expected in determining the section and material properties of beams incorporating precast shells.

18.5.6.2 *Requirements for fully-composite action*

Fully-composite action of the precast shell and cast-in-place core of the beam may be assumed only when the shear stresses along the interface between the precast shell and the cast-in-place core comply with 18.5.4.

In determining the nominal shear stresses on the interface, account shall be taken of transverse and longitudinal shear stresses that may occur.

18.5.6.3 *Design of precast shell*

When designing the precast shell in accordance with 18.5.2 and 18.5.3 consideration shall be given as to whether composite action of the beam can be relied on to resist some or all of the forces applied to the shell or whether by design or through other effects the shell beam shall carry the applied forces alone.

18.5.6.4 *Shear strength of composite beam*

In determining the shear strength of the fully composite beam, rational analysis shall be used to evaluate the contributions to shear resistance of all stirrups and ties and the combined concrete of the precast shell and of the cast-in-place core of the beam.

18.6 Structural integrity and robustness

18.6.1 *Load path to lateral force-resisting systems*

Precast concrete elements shall be connected to other precast elements, cast-in-place concrete or steel elements or to the foundation structure in a manner that ensures that effective load paths for the transfer of forces to primary lateral force-resisting systems can be developed. For the purposes of this clause, a floor system consisting of precast elements and a cast-in-place topping shall be regarded as being precast.

18.6.2 *Diaphragm action*

Where precast components participate in the transfer of horizontal forces by means of diaphragm action, the requirements of 13.3.7.4 shall also be satisfied.

18.6.3 *Wall structures three or more storeys high*

For precast concrete systems supported on precast wall structures three or more storeys high, the following provisions shall apply:

(a) A continuous load path in floor and roof members as required in 18.6.1 shall provide a tensile capacity by way of longitudinal and transverse ties continuous over internal wall supports and between members and external walls. A nominal strength equivalent equal to or greater than 22 kN per metre shall be separately provided along and across the building. Ties parallel to slab spans shall be spaced at not more than 3 m centres. Provisions shall be made to transfer forces around openings;

(b) In addition, continuous reinforcement shall be placed around the perimeter and within 1.2 m of the edges of each floor and the roof, to resist the design forces and to have a nominal strength in tension equivalent to greater than 70 kN.

18.6.4 *Joints between vertical members*

Vertical tension reinforcement across horizontal joints of essential vertical precast structural members shall be provided in accordance with the following requirements:

(a) Precast columns shall have a nominal strength in tension equal to or greater than that corresponding to a reinforcement ratio of $1.5/f_y$;

(b) For columns with a cross-sectional area larger than that required by consideration of loading, the use of a proportionally reduced effective area equal to or greater than one-half of the total area, A_g, may be used to satisfy 18.6.4 (a);

(c) Precast panels shall have continuous vertical tension reinforcement over the full height of the building capable of transmitting the design forces, and shall have a nominal tensile strength equivalent to at least 45 kN per metre of horizontal wall length. Two or more vertical ties shall be provided in each wall panel.

18.6.5 *Connections*

Connections between precast elements, and between precast and cast-in-place concrete elements, shall be designed to meet the following requirements:

(a) To control cracking due to restraint of volume change, and differential temperature gradients;

(b) To develop a failure mode by yielding of steel reinforcement or other non brittle mechanism;

(c) To provide resistance against sliding with sole reliance on friction from gravity loads, except for heavy modular unit structures for which resistance to over turning or sliding has a factor of safety of five or more, or where sliding or rocking will not adversely affect the performance of the structure.

18.6.6 *Frames supporting precast floors*

Frames supporting precast floors shall be tied to the floor in accordance with 10.3.6.

18.6.7 *Deformation compatibility of precast flooring systems*

18.6.7.1 *General*

Precast floor systems shall be designed and detailed to meet the requirements of 2.6.1.1. The implications of the deformation of the primary structure for the seating of the floor system and the integrity of the topping slab shall be considered so that these elements meet their performance requirements.

Design may be based on rational calculation or on methods proved through testing. Calculations or tests shall demonstrate that detailing of the support will permit rotations between the precast floor unit and the
A2 support consistent with 1.5 times the inter-storey drift calculated in accordance with NZS 1170.5 or other referenced loadings standard. Refer also to Clause 19.3.11.2.4 for shear strength of pretensioned floor units near supports.

18.6.7.2 *Precast flooring parallel to beams*

Where hollow-core flooring runs parallel to an adjacent beam which is supported by a column or columns, then either:

A2 (a) The hollow-core unit shall be linked to the parallel beam by the reinforced topping slab only, and shall be placed no closer than the larger of 600 mm or 6 times the thickness of the linking slab to the beam; or

(b) Calculations shall be conducted to demonstrate that deformation incompatibility between the beam and floor at the ultimate limit state will not cause failure of the hollow-core unit.

A2 With other forms of precast floor units a similar link slab shall be provided, or alternatively calculations shall demonstrate that there is sufficient flexibility and ductility in the link between the precast floor unit and adjacent structural elements so that deformation incompatibility cannot lead to failure of the floor units.

18.7 Connection and bearing design

18.7.1 *Transfer of forces between members*

Forces may be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing bar connections, welded or bolted connections reinforced topping, or a combination of these means.

18.7.2 *Adequacy of connections*

The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, the provisions of 7.7 may be applied as appropriate.

18.7.3 *Connections using different materials*

When designing a connection using materials with different structural properties, their relative stiffnesses, strengths, and ductilities shall be considered.

18.7.4 *Floor or roof members supported by bearing on a seating*

For precast floor or roof members supported by bearing onto a seating, with or without the presence of a cast-in-place topping and/or continuity reinforcement, the following requirements shall be satisfied unless shown by analysis or test that alternative details are acceptable:

(a) The support for flooring units seated in potential plastic hinge regions shall meet the requirement of 16.4.3;

(b) Design shall provide for the following minimum seating requirements:

  (i) Each member and its supporting systems shall have design dimensions selected so that, under a reasonable combination of unfavourable construction tolerances, the distance from the edge of the support to the end of the precast member in the direction of its span is at least 1/180 of the clear span but equal to or greater than:

    (A) For solid slabs ..... 50 mm

    (B) For hollow-core slabs, beams or ribbed members ..... 75 mm

  (ii) Bearing pads at unarmoured edges shall be set back a minimum of 15 mm from the edge, or at least the chamfer dimension at chamfered edges.

(c) Where hollow-core units supported on a seating are used in buildings, they shall be mounted at both ends on continuous low friction bearing strips with a coefficient of friction of less than 0.7 and a minimum width of 50 mm;

(d) The seating requirements provided under 18.7.4(b)(i) may be reduced by 15 mm where armoured edges are utilised in the supporting member and adequate support will continue to be provided following plastic hinge formation and elongation;

(e) The requirements of 9.3.9.4.9 and 9.3.9.4.10 shall be satisfied.
(f) In the plastic hinge regions of ductile structures, it shall be assumed that the cover concrete spalls;
(g) Either:
    (i) calculations or tests shall demonstrate that the detailing of the support will permit the rotations between the hollow-core unit and the support greater than 1.5 times the inter-storey drift calculated in accordance with AS/NZS 1170.5.
    (ii) either the support detail described in C18.6.7 (a) to (d) or in (e) shall be used. A2
(h) Bridge spans composed of precast concrete superstructure elements shall satisfy the connection and support overlap requirements of the Transit New Zealand "Bridge Manual".

18.7.5 *Development of positive moment reinforcement*

The requirements of 8.6.13.1 shall not apply to the positive bending moment reinforcement for statically determinate precast members, but at least one-third of such reinforcement shall extend to the centre of the bearing length, taking into account tolerances described in 18.3.

18.8 Additional requirements for ductile structures designed for earthquake effects

18.8.1 *Composite concrete flexural members*

18.8.1.1 *Diaphragm action*

Where diaphragm action is to be provided by means of a cast-in-place topping, the requirements of 13.4.3 shall be satisfied.

18.8.1.2 *Frame dilatancy*

Adequate support shall be provided to precast flooring units to take account of inelastic actions of ductile frames including the effects of frame dilatancy.

18.8.1.3 *Precast shell beam construction*

18.8.1.3.1 *Length of potential plastic hinge regions in moment resisting frames*

For beams of moment resisting frames that are constructed incorporating precast shells and are expected to form plastic hinges, the ductile detailing lengths shall be taken to be equal to or greater than twice the depth of the cast-in-place cores of the beams.

18.8.1.3.2 *Flexural strength in potential plastic regions*

In potential plastic regions the nominal and design flexural strengths shall be determined from the cast-in-place concrete beam core alone.

18.8.1.3.3 *Flexural overstrength*

The flexural overstrength of the potential plastic regions shall be determined as follows:

(a) For moments that induce tension stresses in the bottom fibres of the precast shell:
    (i) When the critical section of a plastic region occurs at the column face or at any distance along the beam for up to the depth of the core away from the column face, the flexural overstrength shall be calculated from the section and material properties of the cast-in-place core of the beam alone;
    (ii) When the critical section of a potential plastic region occurs at a distance greater than the depth of the core along the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.
(b) For moments that induce tension stresses in the top fibres of the beam, the flexural overstrength shall be calculated from the section and material properties of the beam assuming fully composite behaviour.

18.8.1.3.4 *Design strength of shell beam in potential plastic region* A2

In potential plastic hinge regions it shall be assumed that there is no composite action between the cast-in-place core and the adjacent shell when designing the shell in accordance with 18.5.6.3, unless detailing is provided to ensure composite action is maintained by ties which are anchored round longitudinal reinforcement in the shell beam and anchored in the cast in place core. The quantity of this reinforcement shall maintain the shear transfer across the interface by satisfying the requirements of either 18.5.4 or 7.7.

18.8.1.3.5 *Flexural design between potential plastic hinge regions*

In the region between the potential plastic hinge regions, the flexural design may be undertaken assuming fully composite action only when the shear stresses, along the interface between the precast shell and the cast-in-place core comply with 18.5.4.

18.8.2 *Broad categories of precast concrete seismic systems*

18.8.2.1 *Construction incorporating precast concrete*

The construction of seismic moment resisting frames and structural walls incorporating precast concrete elements generally fall into two broad categories, either "equivalent monolithic" systems or, "jointed" systems. The distinction between these types of construction is based on the design of the connections between the precast concrete elements as provided by 18.8.2.2 to 18.8.2.3:

18.8.2.2 *Equivalent monolithic systems*

18.8.2.2.1 *Definition*

A precast concrete structural system satisfying the requirements of this clause shall have strength and toughness equivalent to that provided by a comparable monolithic reinforced concrete structure.

18.8.2.2.2 *Connections in monolithic systems*

The connections between precast concrete elements of equivalent monolithic systems (cast-in-place emulation) can be subdivided into two categories:

(a) *Strong connections of nominal ductility*

In moment resisting frames and structural walls these connections are protected by a capacity design approach which ensures that flexural yielding occurs away from the connection region;

(b) *Ductile connections*

Ductile connections of equivalent monolithic systems typically comprise longitudinal reinforcing bars in the connection which are expected to enter the post-elastic range in a severe earthquake.

In moment resisting frames yield penetration may occur into the connection end-region. The potential plastic hinge region may extend a distance along the end of the member as in cast-in-place construction.

18.8.2.3 *Jointed systems*

18.8.2.3.1 *Definition*

In jointed systems the connections are weaker than the adjacent precast concrete elements. Jointed systems do not emulate the performance of cast-in-place concrete construction. The post-elastic deformations of these systems during an earthquake are typically concentrated at the interfaces of the precast concrete elements where a crack opens and closes.

18.8.2.3.2 *Connections in jointed systems*

The connections between precast concrete elements of jointed systems can be subdivided into three categories:

(a) *Connections of limited ductility*

Connections of limited ductility in jointed systems are usually dry connections formed by welding or bolting reinforced bars or plates or steel embedments and dry-packing and grouting. These connections do not behave as if part of a monolithic construction and generally have limited ductility. An example of a jointed system with connections of limited ductility involving structural walls is tilt up construction. Generally such structures are designed for limited ductility or nominally ductile behaviour;

(b) *Ductile jointed connections*

Ductile connections of jointed systems are generally dry connections in which unbonded post-tensioned tendons are used to connect the precast concrete elements together. The non-linear deformations of the system are concentrated at the interfaces of the precast concrete elements where a crack opens and closes. The unbonded post-tensioned tendons remain in the elastic range. These connections have the advantage of reduced damage and of being self-centring (i.e., practically no residual deformation) after an earthquake;

(c) *Ductile hybrid connections*

Hybrid systems have connections which combine both unbonded post-tensioned tendons and longitudinal steel reinforcing bars (tension/compression yield) or other energy dissipating devices (e.g., flexing steel plates or friction devices).

Appendix B of Part 1 provides some guidance and further references for the design of ductile-jointed hybrid precast concrete systems.

NOTES

19 PRESTRESSED CONCRETE

19.1 Notation

a depth of equivalent rectangular stress block as defined in 7.4.2.7, or depth of compression force against post-tension anchor, mm

A area of concrete between extreme tension fibre and centroid of uncracked section, mm^2

A_c area of concrete at the cross section considered, mm^2, or area of core or spirally confined compression zone measured to outside of spiral, mm^2

A_{cf} larger gross cross-sectional area of the slab-beam strips of the two orthogonal equivalent frames intersecting at a column of a two-way slab, mm^2

A_{cv} effective shear area, area used to calculate shear stress, mm^2 | A2

A_g gross area of section, mm^2

A_{ps} area of prestressed reinforcement in flexural tension zone, mm^2

A_s area of non-prestressed tension reinforcement, mm^2

A_s' area of non-prestressed compression reinforcement, mm^2

A_v area of shear reinforcement within a distance s, mm^2

b width of compression face of member, mm

b_o is perimeter of critical section, mm

b_w web width, mm

c distance from extreme compression fibre to neutral axis, mm

c_c clear cover from the nearest surface in tension to the surface of the flexural tension steel, mm

d distance from extreme compression fibre to centroid of flexural tension reinforcement, but for prestressed members need not be taken as less than $0.8h$, mm

d_c the distance from extreme compression fibre to the centroid of the prestressed reinforcement, mm

d_p distance from extreme compression fibre to centroid of prestressing reinforcement, or to combined centroid of the area of reinforcement when non-prestressing tension reinforcement is included, mm

d' distance from extreme compression fibre to centroid of compression reinforcement, mm

e base of Napierian logarithm

E_c modulus of elasticity of concrete, MPa

E_p modulus of elasticity of prestressing steel, MPa

f_c' specified compressive strength of concrete, MPa | A2

f_{ci}' compressive strength of concrete at time of initial prestress, MPa

f_{dc} stress in a reinforcing bar before concrete cracks when the stress in the concrete surrounding the bar is zero, MPa | A2

f_{px} stress in tendon at distance L_{px} measured from the jacking end, MPa | A2

f_{pe} compressive stress in concrete due to effective prestressing forces only (after allowing for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads, MPa

f_{pi} stress in tendon immediately after transfer, MPa

f_{pj} stress in tendon at the jacking end, MPa

f_{ps} stress in prestressed reinforcement at nominal strength, MPa

f_{pu} tensile strength of prestressing steel being the quotient of the characteristic minimum breaking force and the nominal cross section area, MPa | A2

f_{py} specified yield strength of prestressing steel, or the 0.2 % proof stress, MPa

f_s stress in non-prestressed bonded reinforcement at service loads, MPa

f_{se} effective stress in prestressed reinforcement (after allowance for all prestress losses), MPa

f_{ss} stress induced on extreme tension fibre due to self strain action, MPa

f_{sw} stress sustained at neutral axis due to self strain action, MPa

f_t extreme fibre stress in tension in the precompressed tensile zone, computed using gross or transformed section properties, MPa

f_y lower characteristic yield strength of non-prestressed longitudinal reinforcement, MPa

g_s distance from centre of reinforcing bar to a point on surface of concrete where crack width is being assessed, mm

h overall thickness of member, mm

I second moment of area of section resisting externally applied loads, mm^4

j the time after prestressing, days

k_b coefficient based on bond characteristics of reinforcement

k_4 coefficient dependent on duration of prestressing force

k_5 coefficient dependent on stress in tendon

k_6 function dependent on average annual temperature

L_{px} the length of the tendon from the jacking end to a point at a distance a from that end, mm

M_{cr} bending moment causing flexural cracking at section due to externally applied loads, N mm

M_{max} maximum design bending moment at section due to externally applied loads, N mm

M_o bending moment sustained at decompression of extreme tension fibre, N mm

M^* design bending moment at section at ultimate limit state, N mm

N_c tensile force in the concrete due to service dead load plus live load, N

$N_{n,max}$ axial load strength of member when the external load is applied without eccentricity, that is, when uniform strain exists across section, N

N^* design axial load at the ultimate limit state, N

p ratio of non-prestressed tension reinforcement, A_s/bd

p_p ratio of prestressed reinforcement, A_{ps}/bd_p

P_{su} factored prestressing force at the anchorage device, N

p' ratio of non-prestressed compression reinforcement, A_s'/bd

R a coefficient equal to the ratio of loss of prestress force due to relaxation of the prestressed tendon to the initial prestress force in the tendon at anchorage or after transfer

R_b basic relaxation of tendon, MPa

R_{sc} ratio of loss of prestress force due to relaxation of tendon to the initial prestress force modified to account for the effects of creep and shrinkage in the concrete

s centre-to-centre spacing of flexural tension steel near the extreme tension face, mm. Where there is only one bar or tendon near the extreme tension face, s is the width of the extreme tension face

T temperature, °C

v_c shear stress resisted by concrete, MPa

V shear force, N

V_b shear resisted by concrete in an equivalent reinforced concrete beam, N

V_c nominal shear strength provided by concrete, N

V_{ci} nominal shear strength provided by the concrete when diagonal tension cracking results from combined shear and moment, N

V_{cw} nominal shear strength provided by the concrete when diagonal tension cracking results from principal tensile stress in web, N

V_p vertical component of effective prestressing force at section, N

V_s nominal shear strength provided by the shear reinforcement, N

V^* design shear force at section at ultimate limit state, N

y_t distance from centroidal axis of gross section, neglecting reinforcement, to extreme fibre in tension, mm

α_{tot} sum of the absolute values of successive angular deviations of the prestressing tension over length L_{px}, radians

α_c linear coefficient of expansion of concrete, $°C^{-1}$

α_s factor for determining the shear carried by concrete at columns of two-way prestressed slabs and footings

β_1 factor defined in 7.4.2.7

β_p constant and to compute V_c in prestressed slabs, or an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects

μ coefficient of friction between post-tension cable and prestressing duct

γ_p factor for type of prestressing tendon
= 0.55 for f_{py}/f_{pu} not less than 0.80
= 0.40 for f_{py}/f_{pu} not less than 0.85
= 0.28 for f_{py}/f_{pu} not less than 0.90

Δf_{ps} stress in prestressing steel at service loads based on cracked section analysis less decompression stress, f_{dc} in prestressing steel, MPa

ε_{cc} creep strain in concrete

ε_{cs} shrinkage strain in concrete

λ factor to provide for lightweight concrete (see 7.7.4.3)

ϕ strength reduction factor (see 2.3.2.2)

ϕ_{cc} design creep factor

ω $p\, f_y / f'_c$

ω' $p'\, f_y / f'_c$

19.2 Scope

19.2.1 *General*

Provisions in this section apply to structural members prestressed with wires, strands or bars meeting the requirements of NZS 3109, AS 1311 or AS 1313 or AS/NZS 4672 for prestressing steels.

19.2.2 *Other provisions for prestressed concrete*

The following provisions shall be applied to the design of prestressed concrete members;

(a) Sections 1 to 7 inclusive;
(b) The spacing of non-prestressed and pretensioned reinforcement in 8.3 but excluding 8.3.5;
(c) The development of non-prestressed reinforcement in 8.6;
(d) The detailing of non-prestressed reinforcement in respect to bar bending, welding, development and splicing in 8.4, 8.5, 8.6 and 8.7 respectively
(e) Other provisions where they are specifically noted in chapter 19.

19.3 General principles and requirements

19.3.1 *General design assumptions*

19.3.1.1 *Design requirements*

Members shall meet the requirements for the serviceability and ultimate limit states specified in this Standard. Design shall be based on strength at the ultimate limit state and on behaviour at the serviceability limit state at all stages that may be critical during the life of the structure from the time the prestress is first applied.

19.3.1.2 *Concrete stresses at the serviceability limit state*

Concrete stresses at the serviceability limit state shall not exceed the values given in 19.3.3.5 unless it can be shown by analysis or test that performance of the member will not be impaired.

19.3.1.3 *Secondary prestressing moments*

The moments due to the reactions which are induced by the prestressing forces shall be:

(a) Included in the calculation of stresses and deflections for the serviceability limit state; and
(b) May be excluded in the calculations relating to the required flexural strength of sections at the ultimate limit state where it can be shown that the member has sufficient ductility to accommodate the associated inelastic deformation;
(c) In the design for shear, load cases both with and without secondary moments shall be considered.

19.3.1.4 *Effect of deformations*

Provision shall be made for the effects on parts of the structure or adjoining structure of elastic and plastic deformation and the effects of volume change due to temperature variation, creep and shrinkage of the concrete.

19.3.1.5 *Possibility of buckling*

The possibility of buckling in a member between points where the concrete and the prestressing steel are in contact and of buckling in thin webs and flanges shall be considered.

19.3.1.6 *Section properties*

In computing section properties before bonding of prestressing steel the effect of loss of area due to open ducts shall be considered.

19.3.1.7 *Tendons deviating from straight lines*

Where tendons are subjected to deviations from a straight line, allowance shall be made for the forces caused by these deviations.

19.3.1.8 *Reinforcement for shrinkage and temperature stresses*

Reinforcement for shrinkage and temperature stresses normal to the direction of prestress shall be provided, where appropriate, in accordance with 8.8 or 18.5.1.

19.3.1.9 *Stress concentrations*

Stress concentrations due to prestressing shall be considered in the design.

19.3.1.10 *Unbonded tendons*

Where unbonded tendons are used:

(a) The use of unbonded tendons is permitted provided they are in accordance with NZS 3109, are adequately protected from corrosion in accordance with 19.3.15, and the exposure classification as defined in Table 3.1 is not C or U;

(b) Serviceability requirements shall be in accordance with 19.3.3;

(c) Bonded reinforcement shall be provided in accordance with 19.3.6.7;

(d) The flexural strength shall be computed in accordance with 19.3.6.

19.3.2 *Classification of prestressed members and sections*

Prestressed concrete flexural members and sections shall be classified by their condition at the serviceability limit state as uncracked (Class U), transitional between cracked and uncracked (Class T), or cracked (Class C) based on the computed extreme fibre stress, f_t, at service loads in the precompressed tensile zone assuming an uncracked section as follows:

(a) Class U: Buildings ..... $f_t < 0.7\sqrt{f_c'}$; Bridges..... $f_t < 0.0\sqrt{f_c'}$

(b) Class T: Buildings ..... $0.7\sqrt{f_c'} < f_t \le \sqrt{f_c'}$; Bridges..... $0.0 < f_t < 0.5\sqrt{f_c'}$

(c) Class C: Buildings ..... $f_t > \sqrt{f_c'}$. Bridges..... $f_t > 0.5\sqrt{f_c'}$

A2 Where, on prestressed two-way slab systems in buildings, uniformly distributed loading creates the critical bending moments, the slab system shall be designed as Class U.

Where f_t exceeds $0.7\sqrt{f_c'}$ and the area of a flange on the tension side of the member exceeds the area of a corresponding flange on the compression side of the member, the member shall be designed as Class C.

The location where a member contains a construction joint and $f_t > 0.0$ shall be considered to be class C.

19.3.3 *Serviceability limit state requirements – flexural members*

19.3.3.1 *General*

Members shall meet the requirements at the serviceability limit state for permissible stresses and deflections.

19.3.3.2 *Calculation of stresses in the elastic range*

For investigation of stresses at transfer of prestress, at service loads, and at cracking loads, elastic theory shall be used with the following assumptions:

(a) Strains vary linearly with depth through the entire load range;

(b) At cracked sections, concrete resists no tension.

19.3.3.3 *Section properties*

For the calculation of stresses under service loads, for Class U and Class T flexural members, either gross or transformed uncracked section properties may be used. For Class C flexural members and the sections at construction joints, cracked transformed section properties shall be used.

19.3.3.4 *Deflection*

Deflections of prestressed members at the serviceability limit state shall be calculated in accordance with 6.8.4 and shall satisfy the requirements of 2.4.2.

19.3.3.5 *Permissible stresses in concrete*

19.3.3.5.1 *Permissible stresses in compression*

For prestressed concrete members, stresses in concrete at service loads shall not exceed the following: A2

(a) For class U, T and C flexural members, compression stresses in concrete in the extreme fibres, calculated on the basis of uncracked sections, shall immediately after transfer of prestress not exceed the stress $0.6\,f'_{ci}$;

(b) For class U and class T flexural members, compression stresses in concrete in the extreme fibres, calculated on the basis of uncracked sections, shall not exceed the following:

  (i) After losses of prestress due to prestress plus sustained service load, or normal live load for bridges ..... $0.45\,f'_c$

  (ii) After losses of prestress due to prestress plus total service load, or traffic overload for bridges ..... $0.6\,f'_c$

  (iii) Where a differential temperature case associated with solar radiation on a member is considered, the stress limit given in (a) or (b) above may be increased to the smaller of $0.75\,f'_c$ or the value given in (a) or (b) above with the addition of $0.67\,\alpha_c E_c T$, where T is the increase in temperature on the surface being considered, E_c is the elastic modulus of the concrete, and α_c is the coefficient of thermal expansion of the concrete.

(c) For class C compression stresses in concrete in the extreme fibres after prestress loss, calculated on the basis of transformed cracked sections, shall not exceed the following:

  (i) $0.52\,f'_c$ under sustained service load or normal live load for bridges

  (ii) $0.65\,f'_c$ under total service load conditions or traffic overload for bridges

  (iii) $0.75\,f'_c$ where differential temperature associated with solar radiation is included in the load combination.

19.3.3.5.2 *Permissible stresses in tension – Class U and Class T members*

(a) The extreme fibre tensile stresses in Class U and Class T members under service loads shall not exceed the tensile stress limits for these member classes respectively given in 19.3.2; A2

(b) The tensile stresses in the concrete of Class U and Class T members immediately after prestress transfer (before time-dependent prestress losses) shall not exceed the following, unless reinforcement is added as specified in (c): A2

  (i) Extreme fibre stress in tension, except as permitted in (ii) A1

  Buildings: $0.25\sqrt{f'_{ci}}$ Bridges: $0.25\sqrt{f'_{ci}} < 1.4$ MPa

  (ii) Extreme fibre stress in tension at the ends of simply supported members

  Buildings: $0.5\sqrt{f'_{ci}}$ Bridges: $0.5\sqrt{f'_{ci}} < 2.8$ MPa;

(c) Where the extreme fibre tensile stress in a section immediately after transfer exceeds the appropriate limit given in (b) above, bonded reinforcement shall be provided to resist the total tensile force carried A2

A2 by the concrete, calculated on the basis of uncracked section analysis. This reinforcement may take the form of:

(i) Additional bonded non-prestressed reinforcement, in which case the area shall be sufficient to sustain the force at a stress equal to, or less than, the smaller of 200 MPa or 0.5 f_y.

(ii) Bonded pretension strands or wires, with an area such that the stress increment is equal to or less than 200 MPa, and the resultant stress in the strands or wires does not exceed the limit in 19.3.3.6.1(d).

This force shall be calculated on the basis of uncracked section properties, and the area of this reinforcement shall be sufficient to resist this force based on a stress equal to or less than the smaller of 210 MPa or $0.5f_y$. This reinforcement shall be distributed relatively uniformly across the width of the tensile face of the member and positioned as close to the extreme tensile fibre of the member as practical.

A2 **19.3.3.5.3** *Crack widths for Class C members*

Crack widths for members subjected to serviceability limit state load combinations, but excluding wind or earthquake, shall be controlled by satisfying either (a) or (b) or (c) below:

(a) The tensile stress in the concrete calculated on the basis of uncracked section, is equal to or less than 0.0 at a construction joint, or $0.4\sqrt{f_c'}$ at other locations;

(b) The following two conditions are met:

(i) The tensile stress increment, Δf_s, is less than 250 MPa; and

(ii) The spacing, s, of bonded reinforcement nearer the extreme tension fibre shall not exceed that given by:

$$s = k_b\left[\frac{90000}{\Delta f_s - 50} - 2.5c_c\right] \quad \text{(Eq.19–1)}$$

or

$$s = k_b\left[\frac{70000}{\Delta f_s - 50}\right] \quad \text{(Eq.19–2)}$$

where

c_c is the clear cover distance between the surface of the reinforcement and the surface of the tension member

Δf_s is the change in stress of reinforcement that occurs between the value sustained in the serviceability design load case being considered and the value when the surrounding concrete is decompressed (at zero stress) after all long-term losses have occurred

k_b is equal to 1 for deformed reinforcing bars, $^2/_3$ for strands and $^5/_6$ where a mixture of deformed bars and strands are used

(c) Where limitations are placed on an acceptable crack width, w, in the flexural tension zone of a member, the crack width may be assessed from 2.4.4.6, in which g_s is replaced by g_s/k_b, where k_b is 1.0 for deformed bars and $^2/_3$ for strands, and f_s is replaced by $(\Delta f_s - 50)$. Where Δf_s is less than 150 MPa, the crack width may be assumed to be satisfactory without calculation.

19.3.3.6 *Permissible stresses in prestressed and Non-Prestressed Reinforcement*

A2 **19.3.3.6.1** *Permissible stresses in prestressed and non-prestressed reinforcement*

Tensile stress in prestressing tendons shall not exceed the following:

(a) Due to jacking force ..... 0.94 f_{py}
but not greater than the lesser of 0.80 f_{pu} or the maximum value recommended by the manufacturer of prestressing tendons and anchorages;

(b) Immediately after prestress transfer ..... $0.82 f_{py}$
but not greater than $0.74 f_{pu}$;
(c) Post-tensioning tendons, at anchorages and couplers, immediately after
tendon anchorage ..... $0.70 f_{pu}$
(d) Pretensioned and post tensioned strands after all losses..... ..... $0.8 f_{py}$ A2
or non-prestressed longitudinal reinforcement ..... $0.8 f_y$

19.3.3.6.2 *Permissible service load stress ranges in prestressed and non-prestressed reinforcement*

(a) The stress range due to frequently repetitive live loading in straight prestressing strands or tendons shall not exceed 150 MPa unless justified by a special study;
(b) The stress range due to infrequent live loading in straight prestressed tendons shall not exceed 200 MPa, unless justified by a special study; A2
(c) The stress range in non-prestressed reinforcement shall comply with 2.5.2.

19.3.3.7 *Reinforcement on sides of beams*

If the total depth of a beam is equal to or greater than 1.0 m, skin reinforcement consisting of deformed reinforcement or bonded tendons shall be provided in the side faces as required by 2.4.4.5.

19.3.4 *Loss of prestress in tendons*

19.3.4.1 *General*

The loss of prestress in tendons at any given time shall be taken to be the sum of the immediate loss of prestress and the time-dependent loss of prestress, calculated in accordance with 19.3.4.2 and 19.3.4.3 respectively.

For structures designed to operate above 40 °C, special calculations based on appropriate test data shall be made.

19.3.4.2 *Loss of prestress due to creep and shrinkage*

The loss of stress in prestressing tendons due to shortening of the cables as a result of elastic strains, creep strains in the concrete and shrinkage strains in the concrete, shall be calculated.

19.3.4.2.1 *General*

The immediate loss of prestress shall be estimated by adding the calculated losses of prestress due to elastic deformation of concrete, friction, anchoring and other immediate losses as may be applicable.

19.3.4.2.2 *Loss of prestress due to elastic deformation of concrete*

Calculation of the immediate loss of prestress due to elastic deformation of the concrete at transfer shall be based on the value of modulus of elasticity of the concrete at that age.

19.3.4.2.3 *Loss of prestress due to friction*

The stress variation along the design profile of a tendon due to friction in the jack, the anchorage and the duct shall be assessed as follows in order to obtain an estimate of the prestressing forces at the critical sections considered in the design.

19.3.4.2.4 *Determination of losses*

The extension of the tendon shall be calculated allowing for the variation in tension along its length.

(a) *Friction in the jack and anchorage*
The loss of prestress due to friction in the jack and anchorage shall be determined for the type of jack and anchorage system to be used;
(b) *Friction along the tendon*
Friction loss shall be calculated from an analysis of the forces exerted by the tendon on the duct. In the absence of more detailed calculations the stress in the tendon, (f_{px}), at a distance L_{px}, measured from the jacking end, may be taken as:

$$f_{px} = f_{pj} e^{-\mu(\alpha_{tot} + \beta_p L_{px})} \quad \text{(Eq. 19–3)}$$

where

f_{pj} is the stress in the tendon at the jacking end

e is the base of Napierian logarithms

μ is the friction curvature coefficient for different conditions which, in the absence of specific data and when all tendons in contact in the one duct are stressed simultaneously, may be taken as:

(i) For greased-and-wrapped coating 0.15

(ii) For bright and zinc-coated metal sheathing ..... 0.15 to 0.20

(iii) For bright and zinc-coated flat metal ducts 0.20

(iv) Plastic ducts 0.14

α_{tot} is the sum in radians of the absolute values of successive angular deviations of the prestressing tendon over the length, (L_{px})

β_p is an estimate, in radians per metre (rad/m), of the angular deviation due to wobble effects, which as a first approximation may be taken as:

(i) For sheathing containing tendons other than bars and having an internal diameter:

(A) <50 mm ..... 0.024 to 0.016 rad/m

(B) 50 mm but <90 mm ..... 0.016 to 0.012 rad/m

(C) 90 mm but ≤140 mm ..... 0.012 to 0.008 rad/m

(ii) For flat metal ducts containing tendons other than bars ..... 0.024 to 0.016 rad/m

(iii) For sheathing containing bars and having an internal diameter of 50 mm or less ..... 0.016 to 0.008 rad/m

(iv) For bars of any diameter in a greased-and-wrapped coating ..... 0.008 rad/m

(v) Plastic ducts ..... 0.001 rad/m

L_{px} is the length of the tendon from the jacking end to the point being considered

19.3.4.2.5 *Verification of friction losses*

The magnitude of the friction due to duct curvature and wobble used in the design shall be verified during the stressing operation.

19.3.4.2.6 *Loss of prestress during anchoring*

In a post-tensioned member, allowance shall be made for loss of prestress when the prestressing force is transferred from the tensioning equipment to the anchorage. This allowance shall be checked on the site and any adjustment correspondingly required shall be made.

19.3.4.2.7 *Loss of prestress due to other considerations*

Where applicable, loss of prestress, due to the following, shall be taken into account in design:

(a) Deformation of the forms for precast members;

(b) Differences in temperature between stressed tendons and the actual stressed structures during heat curing of the concrete;

(c) Changes in temperature between the time of stressing the tendons and the time of casting concrete;

(d) Deformations in the construction joints of precast structures assembled in sections;

(e) Relaxation of the tendon prior to transfer.

19.3.4.3 *Time-dependent losses of prestress*

19.3.4.3.1 *General*

The total time-dependent loss of prestress shall be estimated by adding the calculated losses of prestress due to shrinkage of the concrete, creep of the concrete, tendon relaxation, and other considerations as may be applicable.

19.3.4.3.2 *Loss of prestress due to shrinkage of the concrete*

The loss of stress in the tendon due to shrinkage of the concrete shall be based on the free shrinkage strain, ε_{cs}, determined in accordance with 5.2.10. In the absence of more detailed calculations, such as

outlined in Appendix CE, the loss of prestress force shall be taken as $E_p \varepsilon_{cs} A_{ps}$, where ε_{cs} may be modified to allow for the effects of reinforcement.

19.3.4.3.3 *Loss of prestress due to creep of the concrete*

The loss of prestress due to creep of the concrete shall be calculated from an analysis of the creep strains in the concrete. In the absence of more detailed calculations, such as outlined in Appendix CE, and provided that the sustained stress in the concrete at the level of the tendons at no time exceeds 0.5 f'_c, the loss of prestress force due to creep of the concrete may be taken as $E_p \varepsilon_{cc} A_{ps}$, in which ε_{cc} is given by:

$$\varepsilon_{cc} = \phi_{cc} (f_{cc}/E_c) \quad \text{(Eq. 19–4)}$$ A2

where

ϕ_{cc} is the design creep factor, calculated in accordance with 5.2.11

f_{cc} is the sustained stress in the concrete at the level of the centroid of the tendons, calculated using the initial prestressing force prior to any time-dependent losses, together with the sustained long-term loads.

19.3.4.3.4 *Loss of prestress due to tendon relaxation*

This clause applies to the relaxation, at any age and stress level, of low-relaxation wire, low-relaxation strand, and alloy-steel bars.

(a) *Basic relaxation*

The basic relaxation coefficient, R_b, of a tendon after one thousand hours at 20 °C and (0.8 f_{pu}) shall be determined in accordance with AS/NZS 4672; A2

(b) *Design relaxation*

The design relaxation coefficient of a tendon, R, shall be determined from:

$$R = k_4 k_5 k_6 R_b \quad \text{(Eq. 19–5)}$$

where

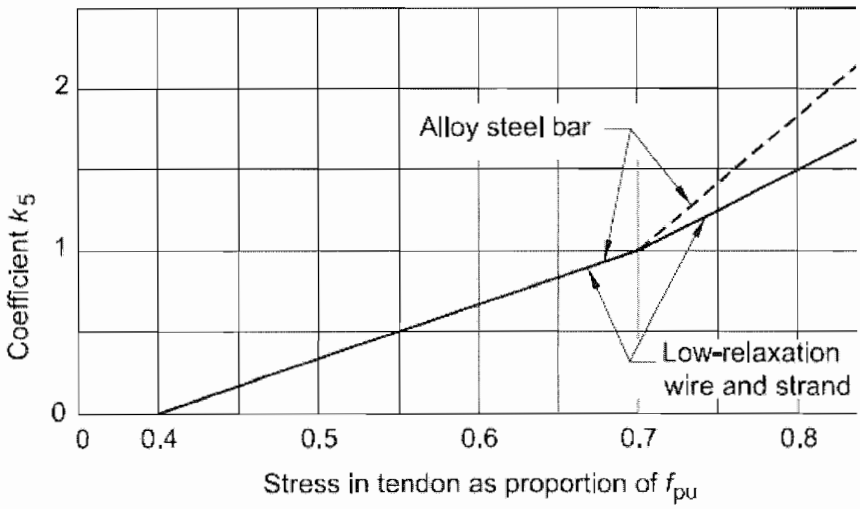
k_4 is a coefficient dependent on the duration of the prestressing force
$= \log [5.4(j)^{1/6}]$

j is the time after prestressing, in days

k_5 is a coefficient, dependent on the stress in the tendon as a proportion of f_p, determined from Figure 19.1

k_6 is a function, dependent on the average annual temperature (T) in °C, taken as $T/20$ but equal to or greater than 1.0.

When determining the design relaxation, consideration shall be given to the effects of curing at elevated temperatures, if applicable.



A2

Figure 19.1 – Coefficient k_5

(c) *Determination of loss due to relaxation*

The proportion of loss of prestress in a tendon due to relaxation of the tendon in the member shall be determined by modifying the stress loss due to the design relaxation of the tendon R, to take into account the effects of shrinkage and creep.

In the absence of more detailed calculations, the proportional loss of stress in the tendon, R_{sc}, in the member may be taken as:

$$R_{sc} = R\left(1 - \frac{\text{the loss of stress due to creep and shrinkage}}{f_{pi}}\right) \quad \text{.....(Eq. 19–6)}$$

where

f_{pi} is the stress in the tendon immediately after transfer.

19.3.4.4 *Loss of prestress due to other considerations*

Account shall be taken, if applicable, of losses due to:

(a) Deformations in the joints of precast structures assembled in sections; and

(b) The effects of any increase in creep caused by frequently repeated loads.

19.3.5 *Ultimate limit state design requirements*

Members shall meet the requirements at the ultimate limit state for flexure axial load and shear.

19.3.6 *Flexural strength of beams and slabs*

19.3.6.1 *Design flexural strength*

The design flexural strength of members containing prestressed reinforcement shall be taken as the nominal strength times the strength reduction factors, given in 2.3.2.2.

19.3.6.2 *Nominal flexural strength*

The nominal flexural strength shall be determined from basic assumptions in 7.4.2 with allowance being made for the additional strain in prestressed reinforcement due to prestressing. The stress in the prestressing tendons at the flexural strength, f_{ps}, shall be determined in accordance with 19.3.6.3, or alternatively where appropriate, it may be determined by the method given in 19.3.6.4.

19.3.6.3 *Strain compatibility analysis*

The stress in prestressed reinforcement in all cases may be determined from strain compatibility analysis using an appropriate stress-strain relationship for the prestressing tendons. In calculating the strain in the prestressing tendons allowance shall be made for strains imposed by prestressing.

19.3.6.4 *Alternative method*

As an alternative to a more accurate determination of f_{ps} based on strain compatibility, the following approximate values of f_{ps} may be used where all the prestressed reinforcement is in the tension zone and if f_{se} is equal to or greater than $0.5f_{pu}$.

(a) For members with bonded tendons:

$$f_{ps} = f_{pu}\left\{1 - \frac{\gamma_p}{\beta_1}\left[p_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right]\right\} \quad \text{.....(Eq. 19–7)}$$

If any compression reinforcement is taken into account when calculating f_{ps} by Equation 19–7, the term

$$\left[p_p \frac{f_{pu}}{f_c'} + \frac{d}{d_p}(\omega - \omega')\right]$$

shall be taken equal to or greater than 0.17 and d' shall be no greater than $0.15d_p$;

(b) For members with unbonded tendons and with a span-to-depth ratio of 35 or less:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{100p_p} \quad \text{.....(Eq. 19–8)}$$

but f_{ps} in Equation 19–8 shall not be taken greater than f_{py} nor greater than $(f_{se} + 420)$;

(c) For members with unbonded tendons and with a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 70 + \frac{f_c'}{300 p_p} \quad \text{(Eq. 19–9)}$$

but f_{ps} in Equation 19–9 shall not be taken greater than f_{py}, nor greater than (f_{se} + 200).

19.3.6.5 *Non-prestressed reinforcement*

Non-prestressed reinforcement if used with prestressing steel, may be considered to contribute to the internal force and to be included in moment strength computations at a stress equal to that determined by strain compatibility analysis.

19.3.6.6 *Limits for longitudinal reinforcement*

19.3.6.6.1 *Maximum amount of reinforcement*

For beams and slabs the amount and distribution of longitudinal prestressed and non-prestressed reinforcement provided shall be such that when the nominal moment of resistance is developed the distance of the extreme compression fibre to the neutral axis shall not exceed the limiting value given in 19.3.6.6.2.

19.3.6.6.2 *Limiting neutral axis depth*

The limiting neutral axis depth shall be calculated from strain compatibility assuming the strain in the concrete in the extreme compression fibre is 0.003 and the increase in tensile strain in the prestressed reinforcement above that sustained when it was initially prestressed, or the strain in non-prestressed reinforcement closest to the extreme tension fibre is 0.0044.

19.3.6.6.3 *Minimum cracking moment*

The design moment in flexure for any section at the ultimate limit state shall be equal to or greater than 1.2 times the moment at first cracking computed on the basis of a modulus of rupture of $0.6\sqrt{f_c'}$. This provision may be waived for:

(a) Two-way, unbonded post-tensioned slabs; and

(b) Flexural members, where the flexural strength is at least twice that required by the ultimate limit state requirements of AS/NZS 1170 and NZS 1170.5 or other referenced loading standard.

19.3.6.6.4 *Placement of bonded reinforcement*

Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the extreme tension fibre in all prestressed flexural members, except that in members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by 19.3.6.7.1 and 19.3.6.7.2.

19.3.6.7 *Minimum bonded reinforcement*

19.3.6.7.1 *Minimum bonded reinforcement with unbonded tendons*

Except for two-way flat slab systems and structures designed in accordance with 19.4.6 the minimum amount of bonded reinforcement, A_s, in members containing unbonded prestressing tendons shall be:

$$A_s = 0.004A \quad \text{(Eq. 19–10)}$$

where A is the area of concrete between the extreme flexural tension face of the member and the centroid of the uncracked section.

The bonded reinforcement shall be uniformly distributed over the pre-compressed tension zone and as close as practicable to the extreme tension fibre. This bonded reinforcement shall be provided regardless of the serviceability limit state stress condition.

19.3.6.7.2 *Minimum bonded reinforcement in two-way flat slab systems with unbonded tendons*

In two-way flat slab systems containing unbonded prestressing tendons, the minimum amount and the distribution of bonded reinforcement, A_s, shall be as follows:

(a) Bonded reinforcement is not required in positive moment areas where the computed concrete tensile stress at the serviceability limit state, after all prestress losses, does not exceed $0.17\sqrt{f_c'}$;

(b) In positive moment areas, where the computed concrete tensile stress at serviceability limit state is greater than $0.17\sqrt{f_c'}$ the minimum area of bonded reinforcement, A_s, shall be:

$$A_s = \frac{N_c}{0.5 f_y} \quad \text{.....(Eq. 19–11)}$$

and f_y shall not exceed 500 MPa. The bonded reinforcement shall be uniformly distributed over the pre-compressed tension zone as close as practicable to the extreme tension fibre;

(c) In negative moment areas at column supports, the minimum area of bonded reinforcement, A_s, in each direction shall be:

$$A_s = 0.00075\, A_{cf} \quad \text{.....(Eq. 19–12)}$$

The bonded reinforcement shall be distributed within a slab width between lines that are 1.5*h* outside opposite column faces, and shall be spaced not greater than 300 mm. At least four bars or wires shall be provided in each direction.

19.3.6.7.3 *Lengths of bonded reinforcement*

Bonded reinforcement required by 19.3.6.7.1 and 19.3.6.7.2 shall have minimum lengths as follows:

(a) Negative moment areas: Sufficient to extend to one sixth of the clear span on each side of the support;

(b) Positive moment areas: One-third of clear span length, centred in the positive moment area;

(c) Where bonded reinforcement is required for flexural strength in accordance with 19.3.6.6 or for tensile stress conditions in accordance with 19.3.6.7.2(b) the anchorage details and development of this reinforcement shall also conform to the provisions of Section 8.

19.3.7 *Compression members – combined flexure and axial loads*

19.3.7.1 *General*

Prestressed concrete members subject to combined flexure and axial load, with or without non-prestressed reinforcement, shall be proportioned by the strength design methods of this Standard. Effects of prestress, creep, shrinkage, and temperature change shall be included.

19.3.7.2 *Axial load limit*

For prestressed columns the design axial load N^*, shall not be taken greater than $0.85\phi N_{n,max}$, where $N_{n,max}$ is the axial load strength at zero eccentricity. In calculating the value of $N_{n,max}$ the strain in the concrete shall not exceed 0.003.

19.3.7.3 *Limits for reinforcement in prestressed compression members*

19.3.7.3.1 *Minimum longitudinal reinforcement*

Members with average prestress f_{pc} less than 1.5 MPa shall have minimum reinforcement in accordance with 10.3.8 and 10.3.9 for columns, or 11.3.11 for walls.

19.3.7.3.2 *Minimum transverse reinforcement*

Columns with an average prestress f_{pc} equal to or greater than 1.5 MPa shall have all tendons enclosed by either spirals or lateral ties in accordance with (a) through (d):

(a) Where spirals are used they shall conform to 10.3.10.5;

(b) Where lateral ties are used they shall be at least 10 mm in diameter and they shall conform to 10.3.10.6;
(c) Ties shall be located longitudinally not more than half a tie spacing above top of footing or slab in any storey, and not more than half a tie spacing below the lowest horizontal reinforcement in members supported above;
(d) Where beams or brackets frame into all sides of a column, ties shall be terminated not more than the smaller of half a tie spacing or 75 mm below lowest reinforcement in such beams or brackets.

19.3.7.3.3 *Minimum transverse reinforcement in walls*

For walls with average prestress f_{pc} equal to or greater than 1.5 MPa, minimum reinforcement required by 11.3.11 need not be applied where structural analysis shows that adequate strength, ductility and stability can be achieved.

19.3.8 *Statically indeterminate structures*

19.3.8.1 *General*

Frames and continuous construction of prestressed concrete shall be designed for satisfactory performance at the serviceability limit state and for adequate strength at the ultimate limit state.

19.3.8.2 *Serviceability limit state*

Performance at the serviceability limit state shall be determined by elastic analysis, considering reactions, moments, shears, and axial forces produced by prestressing, (including secondary prestressing moments) together with serviceability loading cases which shall include any significant self strain loading conditions.

19.3.8.3 *Ultimate limit state*

Design calculations for the ultimate limit state shall ensure the design flexural strength exceeds the design flexural actions. In determining the design actions for flexure, moments may be redistributed as specified in 19.3.9. However, in determining design shear forces, critical actions shall be determined both with and without redistribution of moments.

19.3.9 *Redistribution of design moments for ultimate limit state*

19.3.9.1 *General*

In design calculations for the ultimate limit state design flexural actions in indeterminate prestressed concrete structures, bending moments found from an elastic analysis may be redistributed to the extent indicated in 19.3.9.2, 19.3.9.3 and 19.3.9.4.

19.3.9.2 *Fundamental analysis for moment redistribution*

19.3.9.2.1 *Where moment redistribution permitted*

Bending moments at supports obtained in an elastic analysis may be reduced or increased where an analysis demonstrates that there is adequate ductility in the potential plastic regions to sustain the inelastic rotations associated with the redistribution.

19.3.9.2.2 *Determining rotational capacity*

In determining the rotational capacity of the potential plastic regions, account shall be taken of:
(a) The properties of the concrete, as defined in 5.2;
(b) The stress strain characteristics of prestressed and non-prestressed reinforcement including their strain capacities; as defined in 5.4;
(c) If the prestressed reinforcement is bonded or unbonded.

19.3.9.3 *Exclusion of secondary moments*

Secondary moments may be neglected in determining ultimate limit state design moments where all the reinforcement is bonded and the depth of the neutral axis at the critical section is equal to or less than 0.2 times the effective depth when acted on by the ultimate limit state moment ignoring secondary moments.

19.3.9.4 *Deemed to apply approach for prestressed concrete members*

Design moments obtained from an elastic analysis, which includes secondary moments may be redistributed in accordance with all the following provisions:

(a) The moment at any section in a member derived from an elastic analysis due to a particular combination of design loads may be reduced by up to 20 % of the numerically largest moment given anywhere by the moment envelope for that particular member, covering all appropriate combinations of design load;

(b) Where, as a result of redistribution, the design moment at a support is reduced, the neutral axis depth, c, shall be smaller than:

$$c < \left(0.5 - \frac{\Delta M}{M_{max}}\right) d \quad \text{.....(Eq. 19–13)}$$

where

ΔM is the change in moment at the support between the value found from an elastic analysis, including the secondary prestress moment, and the redistributed value.

M_{max} is the numerically largest bending moment due to the applied loads anywhere in the particular span being considered covering all appropriate combinations of design loads.

(c) The prestressed reinforcement is bonded.

19.3.9.5 *Design moments*

(a) Where moments at the supports of a structure are changed by redistribution, as permitted in 19.3.9.2, 19.3.9.3 or 19.3.9.4, intermediate values shall be adjusted to maintain equilibrium of both vertical and horizontal forces.

(b) The design strength at any section shall not be less than 80 % of the maximum bending moment at that section (including secondary moments) found in an elastic analysis.

19.3.9.6 *Redistribution in members with unbonded prestressed reinforcement*

Where unbonded prestressed reinforcement is used, redistribution of moments shall not be used unless the requirements of 19.3.9.2 are satisfied and any non-prestressed reinforcement is Grade E.

19.3.10 ***Slab systems***

19.3.10.1 *Design actions*

The design moments and shears in prestressed slab systems reinforced for flexure in more than one direction shall be determined in accordance with the provisions of Section 6.

19.3.10.2 *Design strengths*

Design flexural strength of prestressed slabs at every section shall be equal to or greater than the required strength. Design shear strength of prestressed slabs at columns shall be equal to or greater than the required strength.

19.3.10.3 *Service load conditions*

At service load conditions, all serviceability limitations, including limits on deflections, shall be met, with appropriate consideration of the factors listed in 19.3.8.2.

19.3.10.4 *Tendon layout*

In slabs, which are designed for uniformly distributed loads, where prestressed tendons provide the primary flexural reinforcement, the spacing of the tendons required for flexural reinforcement shall be equal to or smaller than the smaller of eight times the slab thickness or 1.5 m. Tendons shall provide a minimum average prestress (after allowance for all prestress losses) of 0.9 MPa on the slab section tributary to the tendon or tendon group. A minimum of two tendons shall be provided in each direction through the critical shear section over columns. Special consideration of tendon spacing shall be provided for slabs with concentrated loads.

19.3.10.5 *Bonded reinforcement*

In slabs with unbonded tendons, bonded reinforcement shall be provided in accordance with 19.3.6.6.4 and 19.3.6.7.

19.3.10.6 *Lift slabs*

In lift slabs with bonded bottom reinforcement and shearheads or lifting collars where it is not practical to pass bottom reinforcement through columns, at least two bonded bottom bars or wires in each direction shall pass through the shearhead or lifting collar as close to the column as practicable and be continued or spliced. At exterior columns the reinforcement shall be anchored at the shearhead or lifting collar.

19.3.11 ***Shear strength***

The design of beams and slabs for shear at the ultimate limit state shall be in accordance with 7.5.

19.3.11.1 *Beams and one-way slabs*

The nominal shear stress, calculated from Equation 7–5 in 7.5.1, shall be equal to or smaller than the smaller of $0.2f_c'$ or 8 MPa, and in calculating the shear stress resisted by concrete f_c' shall not be taken greater than 50 MPa. A2

The area used to calculate the shear stress, A_{cv}, shall:

(a) For rectangular T- and I- section shapes, be taken as the product of the web width, b_w and the effective depth, d;

(b) For octagonal, circular, elliptical and similar shaped sections, be taken as the area of concrete enclosed by the transverse reinforcement;

(c) For hollow-core sections, be taken as the effective depth for the reinforcement resisting the flexural tension force times the minimum web width.

Where composite construction is used to build up a section, the value of A_{cv} and other section properties for any particular increment of loading should be appropriate to the section that exists when the load is applied. Where appropriate, the influence of redistribution of actions within the section due to creep and or shrinkage of concrete on shear strength should be considered.

19.3.11.2 *Nominal shear strength provided by the concrete*

The nominal shear strength provided by concrete shall be assessed either from 19.3.11.2.1 or 19.3.11.2.2, as appropriate, or v_c shall be taken as zero if a strut and tie analysis is used to design the shear reinforcement.

19.3.11.2.1 *Simplified method for determining nominal shear strength of concrete in beams and one-way slabs*

This method of calculating the nominal shear strength of concrete may be used as an alternative to the method given in 19.3.11.2.2, where:

(a) The effective prestress force provides 40 % or more of the nominal flexural strength of the member; A2

(b) The member is not subjected to axial tension or self strain actions, such as differential temperature, which can induce significant tensile stresses over part of the member.

The shear strength provided by the concrete, V_c, is given by:

$$V_c = \left(\frac{\sqrt{f_c'}}{20} + 5\frac{V^* d_c}{M^*} \right) A_{cv} \quad \text{.....(Eq. 19–14)}$$ A2

where

The quantity $(V^* d_c / M^*)$ shall not be taken greater than 1.0, where M^* and V* are the design moment and shear force occurring simultaneously at the section considered, and d_c is the distance from extreme compression fibre to centroid of the prestressed reinforcement;

V_c need not be taken less than $0.14\sqrt{f_c'}\ A_{cv}$, and shall not be taken greater than $0.4\sqrt{f_c'}\ A_{cv}$, except where the critical section lies within the transfer length of a strand or a single wire. Where this occurs the value of V_c shall be based on the calculated prestress level at the critical section assuming the transfer lengths A2

are 50 or 100 diameters for the strand and wire respectively, and the value of V_c shall not exceed the value given by 19.3.11.2.3.

19.3.11.2.2 *General method for determining V_c in beams and one-way slabs*

The nominal shear strength provided by the concrete, V_c, shall be the lesser of the shear force sustained at flexural shear cracking, V_{ci}, as given in (a) and the web-shear force sustained at web-shear cracking, V_{cw}, as given in (b).

(a) The value of V_{ci} is given by:

$$V_{ci} = V_b + \frac{V^* M_o}{M^*} \quad \text{.....(Eq. 19–15)}$$

and M_o is the bending moment corresponding to decompression of the extreme tension fibre under the action of the applied loading, which is given by:

$$M_o = \left(\frac{I}{y_t}\right)\left(f_{pe} + f_{ss}\right) \quad \text{.....(Eq. 19–16)}$$

where

A2 | V_{ci} need not be taken less than $0.14\sqrt{f_c'}\ A_{cv}$

The value of V_b is equal to the value of V_c for a reinforced concrete beam of the same size and reinforcement content as given by 9.3.9.3.4

V^* and M^* are the critical combinations of design shear force and bending moment at the section being considered

f_{ss} is the self strain stress induced on the extreme tension fibre, taken as negative for tension.

(b) The value of V_{cw} is given by:

A2 | $$V_{cw} = 0.3(\sqrt{f_c'} + f_{pc} + f_{sw})\ A_{cv} + V_p \quad \text{.....(Eq. 19–17)}$$

where f_{sw} is the self strain stress sustained at the neutral axis, and f_{pc} is the corresponding longitudinal prestress at the neutral axis, both taken as negative for tension.

Alternatively, V_{cw} may be taken as the shear force that is sustained when the principal tensile stress in the load case being considered, is equal to $0.33\sqrt{f_c'}$ at the centroidal axis of the member, or at the intersection of the flanges with the web when the centroidal axis is in the flange.

19.3.11.2.3 *Shear strength in transfer length*

A2 | In pretensioned members:

(a) Where a section at a distance of $h/2$ from the face of support is closer to the end of the member than the transfer length of the prestressing reinforcement; or

(b) Where bonding of some of the tendons does not extend to the end of the member and the critical section for shear is within the transfer length of the strand or wire.

When computing the value of V_c, which is taken as the lesser of V_{ci} or V_{cw}, the magnitude of the prestress force at the critical section shall be calculated assuming that the force in the pretensioned reinforcement increases linearly over the development length. The development length shall be taken as 50 diameters for strands and 100 diameters for single wires.

At the critical distance of $h/2$ from the face of simple supports where negative moments cannot develop, the value of V_c shall be taken as the smaller of V_{ci} given by Equation 19–15, or the smaller value of V_{cw} given by either Equation 19–17 or $0.4\sqrt{f_c'}\ A_{cv}$.

Specific requirements for precast floor units are given in 19.3.11.2.4. A2

19.3.11.2.4 *Shear strength of pretensioned floor units near supports*

Floors containing precast units, in which negative moments and axial tension can be applied to the units through reinforcement in the concrete topping and/or reinforcement in filled cores, shall be designed to sustain the shear associated with both positive and negative moments acting at the supports.

(a) Pretensioned precast floor units which contain pretensioned strands close to both the upper and lower surfaces of the units, shall be designed to satisfy either the requirements of 19.3.11.2.3, with the value of V_{ci} equal to the larger of that given by 19.3.11.2.1 or 19.3.11.2.2, or the value given in (b) below;

(b) Precast pretensioned floor units which do not contain pretensioned strands close to the top surface of the unit, shall satisfy the requirements of 19.3.11.2.3 for actions associated with positive moments close to the support, and the requirements given below for the region of the floor where reinforcement in the concrete topping and/or close to the top surface of the precast units is subjected to tension due to negative moment and axial tension:

  (i) For units supported by bearing on their lower face, the critical sections for shear in the negative moment region shall be taken as a distance, d, away from the face of the support and at the section at the end of filled cells

  (ii) The design shear strength of the concrete shall be taken as $\phi v_c A_{cv}$, where v_c is given below;

    (A) For hollow-core units with near circular voids and a depth equal to or less than 350 mm
    $v_c = 0.2\sqrt{f_c'} \le 1.43$ MPa

    (B) For hollow-core units where the web width is uniform over a height of ¼ of the depth of the units or more, or the overall depth exceeds 350 mm, v_c is given by 9.3.9.3.4, but with $k_a v_b$ in Equation 9–5 replaced with $k_a v_b = (0.10 + 10 p_w)\sqrt{f_c'} \le 0.2\sqrt{f_c'}$
    where p_w is equal to the area of negative moment reinforcement divided by A_{cv}

    (C) For other forms of pretensioned units or composite unit and *in situ* concrete topping, the value of v_c shall be found from 9.3.9.3.4

    (D) For part (b), d is the effective depth for negative moments taken as the distance from the centroid of non-prestressed reinforcement near the top surface of the floor to the bottom surface of the precast unit.

f_c' is the concrete strength in the precast unit, except where cores are filled in hollow-core units, f_c' shall be taken as the strength of *in situ* concrete

p_w is the proportion of non-prestressed reinforcement near the top surface of the floor divided by A_{cv}.

19.3.11.2.5 *Shear strength in two-way prestressed concrete slabs*

The shear strength of two-way slabs shall be calculated as for 12.7.1, 12.7.2, 12.7.3 and 12.7.4, except the modifications given below may be made:

The value of v_c given in 12.7.3.2 for punching shear may be replaced by the value of v_c given in Equation 19–18.

At columns on slabs or footings where the requirements of 19.3.6.7 are satisfied, the shear stress resisted by the concrete when shear reinforcement is not required, given in 12.7.4.2, may be replaced by the value of v_c given in Equation 19–18.

$$v_c = k_d \beta_p \sqrt{f_c'} + 0.3 f_{pc} + \frac{V_p}{b_o d} \quad \text{.....(Eq. 19–18)}$$ A2

where

β_p is the smaller of 0.29 or $(\frac{\alpha_s d}{b_o} + 1.5)/12$

α_s is 40 for interior columns, 30 for edge columns, and 20 for corner columns

b_o is perimeter of critical section defined in 12.7.1(b)

f_{pc} is the average value of f_{pc} for the two directions; and

A2 k_d allows for the influence of depth on v_c , given by: $k_d = \sqrt{\frac{200}{d}}$ with limits of $1.0 \le k_d \le 0.5$

V_p is the vertical component of all effective prestress forces crossing the critical section.

v_c may be computed by Equation 19–18 if the following are satisfied; otherwise, 12.7.3.2 shall apply:

(a) No portion of the column cross section shall be closer to a discontinuous edge than four times the slab thickness;

(b) f_c' in Equation 19–18 shall not be taken as greater than 35 MPa; and

(c) f_{pc} in each direction shall be equal to or greater than 0.9 MPa, nor be taken greater than 3.5 MPa.

Where shear reinforcement is required the shear stress resisted by the concrete shall be equal to $0.17\sqrt{f_c'}$.

A2 **19.3.11.2.6** *Shear resisted by beam type action in two-way slab*

The shear strength of a prestressed footing slab under beam type action in the vicinity of concentrated loads or reactions shall be calculated for a critical section perpendicular to the plane of the slab, extending across the entire width and located at a distance of *h*/2 from the face of the concentrated load or reaction. For this condition, the slab or footing shall be designed in accordance with 7.5 and 19.3.11.1, 19.3.11.2 and 19.3.11.3.

19.3.11.3 *Nominal shear strength provided by shear reinforcement*

19.3.11.3.1 *Details of shear reinforcement in slabs*

Shear reinforcement consisting of bent up bars or stirrups, shall not be assumed to contribute to shear strength in one- or two-way slabs unless either:

(a) The effective depth of the slab is equal to or greater than the smaller of 150 mm or 16 times the diameter of the stirrup; or

(b) The stirrup is anchored mechanically on the compression surface of the slab.

19.3.11.3.2 *Critical section for shear in prestressed members*

For prestressed members, sections located at less than a distance *h*/2 from face of support shall be
A2 designed for the same shear, V^*, as that computed at a distance *h*/2 provided the conditions in 9.3.9.3.1 are satisfied.

19.3.11.3.3 *Shear strength provided by reinforcement*

Shear reinforcement in prestressed concrete members shall be designed in accordance with 7.5.5, 7.5.6, 7.5.7, 7.5.8 and 7.5.9 and in accordance with the appropriate clauses given in (a), (b), (c) or (d) below: .

(a) Beams and one-way slabs shall satisfy Equation 9–6 and 9.3.9.3 with the modifications noted in 19.3.11.3.4;

(b) For columns and piers, 10.3.10.4;

(c) For walls,11.3.10.3.8;

(d) Two-way slabs, 12.7.4.

19.3.11.3.4 *Modification of design of shear reinforcement in beams and one-way slabs due to prestress*

In the design of shear reinforcement in beams and one-way slabs where the effective prestress force is equal to or greater than 40 % of the tensile strength of flexural tension reinforcement, the following modifications shall be made to 9.3.9.4.12 and 9.3.9.4.15:

(a) When 9.3.9.4.12 is applied the maximum spacing limits for shear reinforcement in part (a) may be increased to the smaller of 0.75*h* or 600 mm.

(b) When 9.3.9.4.15 is applied and shear reinforcement is required by 9.3.9.4.13, the minimum area of shear reinforcement, A_v, shall be the smaller of that given by Equation 9–10 or by Equation 19–19 below.

A2

$$A_v = \frac{A_{ps} f_{pu} s}{80 f_y d} \sqrt{\frac{d}{b_w}} \quad \text{.....(Eq. 19–19)}$$

where s is the spacing of transverse reinforcement in direction parallel to the longitudinal reinforcement, mm.

19.3.12 ***Torsional strength***

The requirement of 7.6 shall be satisfied with the following changes:

(a) In 7.6.1.2 the value of $0.1\phi A_{co} t_c \sqrt{f_c'}$ may be replaced by:

$$0.1\phi A_{co} t_c \sqrt{f_c'} \left(1 + \frac{f_{pc}}{0.33\sqrt{f_c'}} \right) \quad \text{.....(Eq. 19–20)}$$

where f_{pc} is the prestress stress after losses acting at the neutral axis of the section, or at the intersection of the flange and web where the neutral axis is located in the flange.

(b) In 7.6.1.3(a), torsional reinforcement is not required if the torsional design action, T^*, calculated from an analysis based on gross section properties, is equal to or less than $0.1\phi A_{co} t_c \sqrt{f_c'} \left[1 + \frac{f_{pc}}{0.33\sqrt{f_c'}} \right]$.

19.3.13 ***Anchorage zones for post-tensioned tendons***

19.3.13.1 *General*

19.3.13.1.1 *Definition of anchorage zone*

The anchorage zone is the portion of the member through which the concentrated prestressing force is transferred to the concrete and distributed more uniformly across the section. Its extent is generally equal to the largest dimension of the cross section. For anchorage devices located away from the end of the member, the anchorage zone includes the disturbed regions both ahead of and behind the anchorage device.

19.3.13.1.2 *Design of anchorage zones*

Anchorage zones shall be designed to sustain local compression stresses bearing against the anchor, and tension forces, which are associated with the transmission of the force in the cable into the member, as detailed in 19.3.13.4.

In the design the following effects shall be considered;

(a) The effect of abrupt changes in section in the anchorage zone;
(b) The three dimensional aspect to the flow of forces requires splitting and spalling forces to be sustained in two planes at right angles;
(c) The sequence of stressing of the cables.

19.3.13.2 *Design forces in prestress tendons*

Design of anchorage zones shall be based upon the factored prestressing force, P_{su}, taken as 1.2 times the maximum prestressing jacking force and a strength reduction factor ϕ of 0.85.

19.3.13.3 *Design material strengths*

19.3.13.3.1 *Tensile strength of bonded reinforcement*

Tensile strength of bonded reinforcement is limited to f_y for non-prestressed reinforcement and to f_{py} for prestressed reinforcement. Tensile stress of unbonded prestressed reinforcement for resisting tensile forces in the anchorage zone shall be limited to $f_{ps} = f_{se} + 70$.

19.3.13.3.2 *Bearing stress against anchors*

The bearing stress in the concrete against the prestressed anchors shall comply with 16.3, except that the
A2 | concrete strength at the time the tendons are stressed, f'_{ci}, shall be used in place of the 28 day design strength, f'_c. The required concrete strength at the time the cables are stressed shall be given on the drawings.

19.3.13.3.3 *Tensile strength of concrete*

In the design of reinforcement to carry bursting and spalling tension forces the tensile strength of concrete shall be neglected.

19.3.13.4 *Design methods*

19.3.13.4.1 *Permitted methods*

The following methods shall be permitted for the design of anchorage zones provided that the specific procedures used result in prediction of strength in substantial agreement with results of comprehensive tests:

(a) Equilibrium based plasticity models (strut-and-tie models);
(b) Linear stress analysis (including finite element analysis or equivalent); or
(c) Simplified methods where applicable.

19.3.13.4.2 *Simplified and linear elastic methods*

Simplified methods may only be used where the method specifically allows for the cross section shape and any change in this shape, which occurs within the anchorage zone. Two-dimensional linear elastic methods (finite element) may be used provided allowance is made for any changes in section dimensions within the anchorage zone. Where simplified or two-dimensional elastic methods are used, analyses shall be made of actions on two axes at right angles to determine reinforcement required to sustain the spalling and bursting forces in each direction.

19.3.13.4.3 *Reinforcement required for tension forces in anchorage zones*

Reinforcement shall be provided to:

(a) Resist bursting forces in anchorage zones;
(b) Control spalling cracks, where these are induced by compatibility;
(c) Resist spalling forces where these are required for equilibrium;
(d) Resist splitting forces in anchorage zones located away from the end of a member, as specified in 19.3.13.4.4.

19.3.13.4.4 *Anchorage devices away from end of members*

For anchorage devices located away from the end of the member, bonded reinforcement with a nominal strength equal or greater than $0.35P_{su}$ shall be provided to transfer the force into the concrete section behind the anchor. Such reinforcement shall be placed symmetrically around the anchorage devices and shall be fully developed both behind and ahead of the anchorage devices.

19.3.13.4.5 *Minimum reinforcement for spalling*

Except where extensive testing or analysis indicates that spalling reinforcement is not required, a minimum reinforcement with a nominal tensile strength equal to 2 % of each factored prestressing force shall be provided in orthogonal directions parallel to the back face of all anchorage zones to control spalling cracks.

19.3.13.5 *Detailing requirements*

Selection of reinforcement sizes, spacings, cover, and other details for anchorage zones shall make allowances for tolerances on the bending, fabrication, and placement of reinforcement, for the size of aggregate, and for adequate placement and consolidation of the concrete.

19.3.14 *Curved tendons*

Where tendons are curved in either plan or elevation, the influence of the radial force that the cable applies to the concrete shall be considered. Where required by analysis reinforcement shall be provided to resist the tensile forces associated with local bending, shear and bursting in the concrete.

19.3.15 *Corrosion protection for unbonded tendons*

19.3.15.1 *General*

Unbonded prestressing steel shall be encased with sheathing. The prestressing steel shall be completely coated and the sheathing around the prestressing steel filled with suitable material to inhibit corrosion.

19.3.15.2 *Watertightness*

Sheathing shall be watertight and continuous over the entire length to be unbonded.

19.3.15.3 *Corrosive environments*

For applications in corrosive environments, the sheathing shall be connected to all stressing, intermediate and fixed anchorages in a watertight fashion.

19.3.16 *Post-tensioning ducts*

19.3.16.1 *General*

Ducts for grouted tendons shall be mortar-tight and non-reactive with concrete, prestressing steel, grout, and corrosion inhibitor.

19.3.16.2 *Single wire, strand or bar*

Ducts for grouted single wire, single strand, or single bar tendons shall have an inside diameter at least 5 mm larger than the prestressing steel diameter.

19.3.16.3 *Multiple wire, strand or bar*

Ducts for grouted multiple wire, multiple strand, or multiple bar tendons shall have an inside cross-sectional area of at least two times the cross-sectional area of the prestressing steel.

19.3.17 *Post-tensioning anchorages and couplers*

19.3.17.1 *Strength of anchorages and couplers*

Anchorages and couplers for bonded and unbonded tendons shall develop at least 95 % of the specified breaking strength of the prestressing steel, when tested in an unbonded condition, without exceeding anticipated set. For bonded tendons, anchorages and couplers shall be located so that 100 % of the specified breaking strength of the prestressing steel shall be developed at the critical sections after the prestressing steel is bonded in the member.

19.3.17.2 *Location of couplers*

Couplers shall be placed in areas approved by the design engineer and enclosed in housing long enough to permit necessary movements. A2

19.3.17.3 *Fatigue of anchorages and couplers*

In unbonded construction subject to repetitive loads, special attention shall be given to the possibility of fatigue in anchorages and couplers.

19.3.17.4 *Protection against corrosion*

Anchorages, couplers, and end fittings shall be permanently protected against corrosion.

19.3.18 *External post-tensioning*

19.3.18.1 *General*

Post-tensioning tendons may be external to any concrete section of a member. The strength and serviceability design methods of this code shall be used in evaluating the effects of external tendon forces on the concrete structure.

19.3.18.2 *Flexural strength*

External tendons shall be considered as unbonded tendons when computing flexural strength unless provisions are made to effectively bond the external tendons to the concrete section along its entire length.

19.3.18.3 *Attachment to member*

External tendons shall be attached to the concrete member in a manner that maintains the desired eccentricity between the tendons and the concrete centroid throughout the full range of anticipated member deflection.

19.3.18.4 *Protection against corrosion*

External tendons and tendon anchorage regions shall be protected against corrosion, and the details of the protection method shall be indicated on the drawings or in the project specifications.

19.4 Additional design requirements for earthquake actions

19.4.1 *General*

This clause covers the design of prestressed and partially prestressed concrete members of ductile moment resisting frames and joints between such members.

19.4.2 *Materials*

19.4.2.1 *Prestressing steel*

The strain in the prestressed reinforcement at ultimate limit state, allowing for the required curvature calculated using the effective plastic hinge length in 2.6.1.3.3, shall not exceed the minimum specified ultimate strain.

19.4.2.2 *Concrete*

The value of f_c' used in design shall not exceed 70 MPa.

19.4.2.3 *Grouting of tendons*

Post-tensioned tendons in moment resisting frame members shall be grouted, except as allowed by 19.4.5.2, or for hybrid structures designed in accordance with 19.4.6.

A2 | ### 19.4.3 *Beams and floor slabs*

19.4.3.1 *Dimensions*

Dimensions of prestressed beams shall be in accordance with the provisions of 9.4.1.

19.4.3.2 *Redistribution of moments*

Provided the limits to flexural steel are in accordance with 19.4.3.3(b) or (c), bending moments derived from elastic analyses may be redistributed in accordance with the provisions of 19.3.9.4 and 19.3.9.5 and secondary moments may be neglected.

19.4.3.3 *Nominally ductile, limited ductile and ductile plastic regions*

Design of regions of various ductility classification shall be limited as follows:

(a) *Nominally ductile plastic regions*

A2 | Permissible curvature in nominally ductile plastic regions shall be calculated from the material strain limits given in 2.6.1.3.4(a) by replacing f_y by $(f_{py}\text{-}f_{so})$, where f_{so} is the stress after losses in the prestress tendon closest the extreme tension fibre when the stress in that fibre is zero.

(b) *Limited ductile plastic regions*

In limited ductile plastic regions the following criteria shall be satisfied:

(i) The depth of the neutral axis at ultimate shall not exceed $0.2h$;

(ii) In rectangular beams, or in T- or L- beams where the compression zone is on the opposite side the flange, the area of compression reinforcement times its yield stress ($A_s' f_y$) shall be equal to or greater than 0.15 times the compression force;

(iii) Transverse reinforcement shall comply with 9.4.5 but with a spacing equal to or less than six times the diameter of the longitudinal bar being held against buckling.

(c) *Ductile plastic regions*

An analysis shall be made based on engineering principles to demonstrate that the curvature ductility is equivalent to that of a similar sized reinforced concrete beam with a ductile plastic region.

19.4.3.4 *Contribution of reinforcement in flanges to strength of beams*

The contribution of reinforcement in a flange of a beam to the design strength shall be determined as set out in 9.4.1.6.1.

In the determination of flexural overstrength the reinforcement in a flange of a beam shall be determined as set out in 9.4.1.6.2.

Where less than 70 % of the longitudinal reinforcement required for the design tensile strength passes through the column the joint zone shall be designed using a strut and tie model. A2

19.4.3.5 *Transverse reinforcement*

Stirrup ties shall be provided in potential plastic hinge regions in accordance with the provisions of 9.4.4.

In potential plastic hinge regions the shear strength provided by the concrete shall be assumed to be zero. Stirrup ties shall be equal to or greater than 10 mm diameter and the distance between vertical legs of stirrup ties across the section and along the beam shall not exceed 200 mm between centres in each set of stirrup ties.

19.4.3.6 *Floors with precast pretensioned units* A2

The precast floor units shall be designed by capacity design to sustain the over-strength actions associated with seismic actions (both horizontal and vertical), together with associated gravity loads, when the reinforcement connecting the units to the supporting structure sustains a stress of $\phi_{o,fy} f_y$ where $\phi_{o,fy}$ is defined in 2.6.5.5.

The supports to the precast units shall be detailed to prevent the transmission of positive moments into the precast unit, due to relative rotation between the support and unit which could lead to flexural failure within the development length of pretensioned or non-prestressed reinforcement.

19.4.4 ***Design of columns and piles***

19.4.4.1 *Confinement and anti-buckling reinforcement*

Design of nominally ductile prestressed piles and columns, and prestressed piles and columns containing potential limited ductile plastic regions, shall satisfy the provisions of 10.3.10.5 or 10.3.10.6, as appropriate. Prestressed concrete piles and columns containing potential ductile plastic regions shall satisfy the provisions of 10.4.7.

19.4.4.2 *Minimum reinforcement content*

In a column containing a ductile or limited ductile plastic region the proportion of reinforcement, p, which includes both the prestressed reinforcement and non-prestressed reinforcement, shall be equal to or greater than $0.5\frac{\sqrt{f_c'}}{f_y}$, where f_y is the yield stress of reinforcement, but with prestressed reinforcement f_y shall not be taken greater than 500 MPa.

19.4.4.3 *Spacing of longitudinal reinforcement*

The spacing of longitudinal prestressed or non-prestressed reinforcement in potential plastic hinge regions shall:

(a) For nominally ductile columns and columns containing potential limited ductile plastic regions, satisfy the provisions of 10.3.8.2 and 10.3.8.3;

(b) For columns containing potential ductile plastic regions satisfy the provisions of 10.4.6.2 and 10.4.6.3.

Where longitudinal reinforcing bars are also utilised as vertical shear reinforcement in beam column joint cores, the distribution of bars shall be in accordance with 15.4.5.3.

19.4.4.4 *Transverse reinforcement in potential plastic regions*

Special transverse reinforcement in accordance with 10.4.7 shall be provided in the regions of pretensioned piles in which ductility is required. The centre-to-centre spacing of spirals shall be equal to or less than 0.25 times the pile width or diameter, or six times the diameter of the longitudinal strand or 200 mm, whichever is least. Shear strength provided by the concrete shall be assumed to be zero.

19.4.4.5 *Shear strength*

Shear strength requirements shall be in accordance with 9.4 for beams and 10.4 for columns.

19.4.5 *Prestressed moment resisting frames*

19.4.5.1 *Beam tendons at beam column joints*

Except as provided by 19.4.5.2, and for structures designed in accordance with 19.4.6, the beam prestressing tendons which pass through joint cores shall be spaced at the face of the columns so that at least one tendon is centred at not more than 150 mm from the beam top and at least one at not more than 150 mm from the beam bottom.

19.4.5.2 *Partially prestressed beams*

For partially prestressed beams in which the non-prestressed reinforcement provides at least 80 % of the design moment for earthquake plus gravity load combinations, prestress may be provided by one or more tendons passing through the joint core and located within the middle third of the beam depth, at the face of the column. In such cases post-tensioned tendons may be ungrouted, provided anchorages are detailed to ensure that neither anchorage failure or cable de-tensioning can occur under seismic actions.

19.4.5.3 *Ducts for grouted tendons*

Ducts for post-tensioned grouted tendons through beam column joints shall be corrugated, or shall provide equivalent bond characteristics. Corrugated ducts are not required for ungrouted tendons complying with 19.4.5.2 or for structures designed in accordance with Appendix B of Part 1.

19.4.5.4 *Jointing material*

Precast members may be connected at beam column joints provided that the jointing material has sufficient strength to withstand the compressive and transverse forces to which it may be subjected. The interfaces shall be roughened or keyed to ensure good shear transfer and the retention of the jointing material after cracking.

19.4.5.5 *Joint reinforcement*

Design of joint reinforcement shall be in accordance with the provisions of 15.4.

Post-tensioning anchors shall only be located in exterior beam column joint cores if it can be demonstrated that the joint can resist both the anchorage tensile bursting stresses and the diagonal tension from beam and column forces.

19.4.6 *Design of hybrid jointed frames*

Hybrid jointed frames shall be designed in accordance with Appendix B of Part 1.

APPENDIX A – STRUT-AND-TIE MODELS (Normative)

A1 Notation

a_1, a_2, a_3 dimensions of nodal zones, mm

a shear span, equal to the distance between a load and a support in a structure, mm

A_c the effective cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut, mm^2

A_n area of a face of a nodal zone or a section through a nodal zone, mm^2

A_{ps} area of prestressed reinforcement in a tie, mm^2

A_{si} area of surface reinforcement in the ith layer crossing a strut, mm^2

A_{st} area of non-prestressed reinforcement in a tie, mm^2

A_s' area of compression reinforcement in a strut, mm^2

b thickness of concrete member forming a strut, mm

d distance from extreme compression fibre to centroid of longitudinal tension reinforcement, mm

f_c' specified compressive strength of concrete, MPa

f_{cu} effective compressive strength of concrete in a strut or a nodal zone, MPa

f_{py} specified yield strength of prestressing steel, or the 0.2 % proof stress, MPa

f_s design steel tensile stress less than the lower characteristic yield strength for non-prestressed reinforcement, MPa

f_s' stress in compression reinforcement, MPa

f_{se} effective stress after losses in prestressed reinforcement, MPa

f_y specified yield strength of non-prestressed reinforcement, MPa

F_n nominal strength of a strut, tie, or nodal zone, N

F_{nn} nominal strength of a face of a nodal zone, N

F_{ns} nominal strength of a strut, N

F_{nt} nominal strength of a tie, N

F^* factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model at ultimate limit state, N

s_i spacing of reinforcement in the ith layer adjacent to the surface of the member, mm

w_s effective width of strut, mm

α_1 factor defined in 7.4.2.7

β_s factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut

β_n factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone

γ_i angle between the axis of a strut and the bars in the ith layer of reinforcement crossing that strut

γ_1, γ_2 angle between strut and reinforcement

Δf_p increase in stress in prestressing tendons due to factored loads, MPa

λ correction factor related to the unit weight of concrete, see 7.7.4

ϕ strength reduction factor

A2 Definitions

The following definitions are additional to those given in Section 1.

B-REGION. A portion of a member in which the plane sections assumption of flexure theory from 7.4.2.2 can be applied.

DISCONTINUITY. An abrupt change in geometry or loading.

D-REGION. The portion of a member within a distance equal to the member height h or depth d from a force discontinuity or a geometric discontinuity.

DEEP BEAM. See 9.3.10.1.

NODE. The point in a strut-and-tie model where the axes of the struts, ties, and concentrated forces acting on the joint intersect.

NODAL ZONE. The volume of concrete around a node that is assumed to transfer strut-and-tie forces through the node.

STRUT. A compression member in a strut-and-tie model. A strut represents the resultant of a parallel or a fan-shaped compression field.

BOTTLE-SHAPED STRUT. A strut that is wider at mid-length than at its ends.

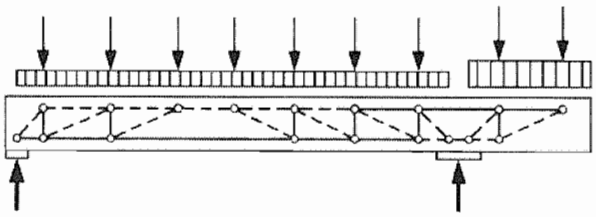
STRUT-AND-TIE MODEL. A truss model of a structural member, or of a D-region in such a member, made up of struts and ties connected at nodes, capable of transferring the factored loads to the supports or to adjacent B-regions.

TIE. A tension member in a strut-and-tie model.

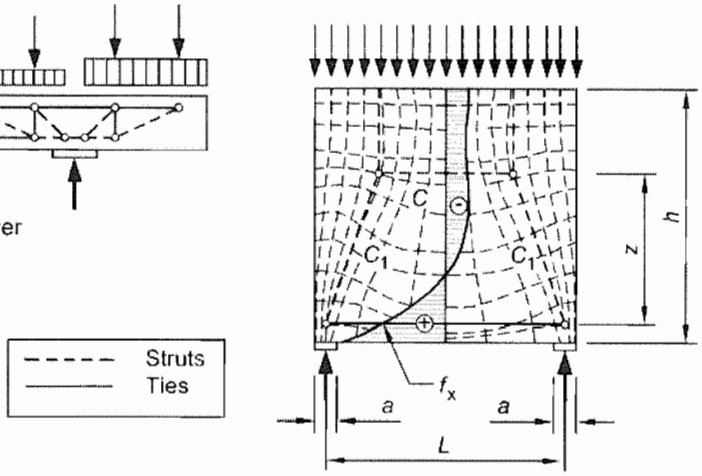
A3 Scope and limitations

A3.1 *General*

Strut-and-tie models are useful when designing regions of reinforced concrete structures where the theory of flexure based on a linear strain distribution does not apply. Examples are deep beams, regions of discontinuity or where high concentrated forces are applied, brackets, corbels and diaphragms or walls with openings. It is a relatively simple technique in which the designer is required to establish an admissible path for internal forces in equilibrium with factored external loads and reactions. Struts, consisting primarily of concrete, are assigned compression forces and ties consisting of reinforcing bars, are the tension members. Struts and ties are joined at nodes and the strut and tie model represents an idealised truss. Simplified stress trajectories, to be simulated by struts and ties, are shown in Figure A.1 for regions of discontinuity.

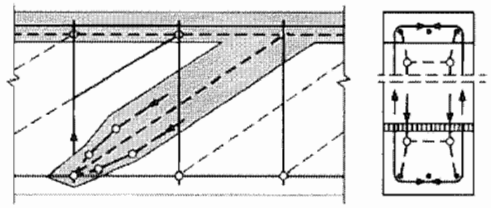


(a) Simple span with cantilever

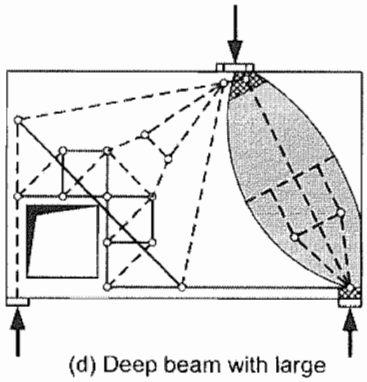


(b) Elastic stress field and strut-and-tie model for a deep beam

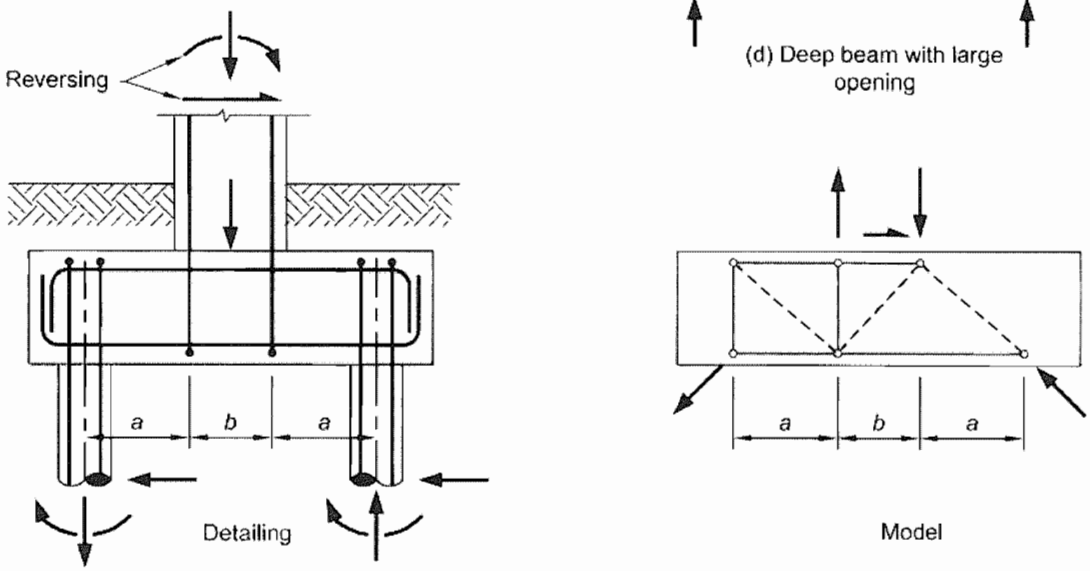
Struts
Ties



(c) Diagonal strut in the web with stirrups



(d) Deep beam with large opening



(e) Pile cap

Figure A.1– Truss models with struts and ties simulating stress trajectories

A3.2 *Nodal zones*

Joints of strut-and-tie models are nodal zones where multi-directional stresses, satisfying equilibrium requirements for each node, need to be transferred.

A3.3 *Dimensions of nodal zones*

With rational approximations, the selected dimensions for nodal zones shall govern the relevant dimensions of adjacent struts and ties. Nodal dimensions shall be used to ascertain that the strength limitations of A7 are not exceeded.

A3.4 *Serviceability limit state*

Recommended strengths for the ultimate limit state given in A5, and A7 are such that performance within the serviceability limit state may be considered to have been satisfied. However, control of possible secondary cracking should be considered in accordance with A5.3.

A4 Strut-and-tie model design procedure

A4.1 *Truss models*

Structural concrete members, or D-regions in such members, may be designed by modelling the member or region as an idealised truss. The truss model shall contain struts, ties, and nodes as defined in A2. The truss model shall be capable of transferring all factored loads to the supports or adjacent B-regions.

A4.2 *Equilibrium requirement*

The strut-and-tie model shall be in equilibrium with the factored applied loads and the reactions.

A4.3 *Geometry of truss*

In determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.

A4.4 *Ties may cross struts*

Ties may cross struts. Struts shall cross or overlap only at nodes.

A4.5 *Minimum angle between strut and tie*

The angle between the axes of any strut and any tie entering a single node shall be equal to or greater than 25°. Where a single strut is used a larger angle shall be used.

A4.6 *Design basis*

Design of struts, ties, and nodal zones shall be based on:

$$\phi F_n \geq F^* \quad \text{(Eq. A–1)}$$

where F^* is the force in a strut or tie, or the force acting on one face of a nodal zone, due to the factored loads; F_n is the nominal strength of the strut, tie, or nodal zone; and ϕ is the strength reduction factor specified in 2.3.2.2.

A5 Strength of struts

A5.1 *Strength of strut in compression*

The nominal compressive strength of a strut without longitudinal reinforcement shall be taken as the smaller value at the two ends of the strut of:

$$F_{ns} = f_{cu} A_c \quad \text{(Eq. A–2)}$$

where

A_c is the cross-sectional area at one end of the strut, and

f_{cu} is the effective compressive strength of the concrete in the strut given in A5.2;

A5.2 Effective compressive strength of concrete strut

The effective compressive strength of the concrete in a strut shall be taken as:

$f_{cu} = \beta_s \alpha_1 f_c'$ ..... (Eq. A–3)

where

$\alpha 1$ is given by 7.4.2.1(c); and

β_s is given by:

(a) For a strut of uniform cross-sectional areas over its length $\beta_s = 1.0$;

(b) For struts located such that the width of the mid-section of the strut is larger than the width at the nodes (bottle-shaped struts):

(i) With reinforcement satisfying A5.3 ..... $\beta_s = 0.75$

(ii) Without reinforcement satisfying A5.3 ..... $\beta_s = 0.60\ \lambda$
where λ is
1.0 for normal weight concrete,
0.85 for sand lightweight concrete and
0.75 for lightweight concrete.

(iii) For struts in tension members, or the tension flanges of members ..... $\beta_s = 0.40$

(iv) For all other cases ..... $\beta_s = 0.60$

A5.3 *Reinforcement for transverse tension*

If the value of β_s specified in A5.2(b)(i) is used, the axis of the strut shall be crossed by reinforcement proportioned to resist the transverse tensile force resulting from the compression force spreading in the strut. It may be assumed that the compressive force in the strut spreads at a slope of 2.5 longitudinal to one transverse to the axis of the strut.

A5.3.1 *Minimum reinforcement*

For f_c' not greater than 40 MPa, the requirement of A5.3 may be satisfied by the axis of the strut being crossed by layers of reinforcement that satisfy:

$$\sum \frac{A_{si}}{bs_i} f_y \sin\gamma_i \geq 1.5\,\text{MPa} \quad \text{..... (Eq. A–4)}$$

where A_{si} is the total area of reinforcement at spacing s_i in a layer of reinforcement with bars at an angle γ_i to the axis of the strut.

A5.3.2 *Placement of reinforcement*

The reinforcement required in A5.3 shall be placed in either:

(a) Two orthogonal directions at angles γ_1 and γ_2 to the axis of the strut; or

(b) In one direction at an angle γ_1 to the axis of the strut.

If the reinforcement is in only one direction, γ_1 shall be equal to, or greater than 40°.

A5.4 *Increased strength of strut due to confining reinforcement*

If documented by tests and analyses, an increased effective compressive strength of a strut due to confining reinforcement may be used.

A5.5 *Increased strength of strut due to compression reinforcement*

The use of compression reinforcement may increase the strength of a strut. Compression reinforcement shall be properly anchored, parallel to the axis of the strut, located within the strut, and enclosed in ties or spiral satisfying 10.4.7.4 and 10.4.7.5 as appropriate. In such cases, the strength of a longitudinally reinforced strut is:

$F_{ns} = f_{cu}A_c + A_s' f_s'$ ..... (Eq. A–5)

A6 Strength of ties

A6.1 Nominal strength of tie

The nominal strength of a tie shall be taken as:

$F_{nt} = A_{st}f_y + A_{ps}(f_{se} + \Delta f_p)$ ..... (Eq. A–6)

where $(f_{se} + \Delta f_p)$ shall not exceed f_{py}, and A_{ps} is zero for non-prestressed members. Un-stressed pretension strand should have an overall limiting value of $\Delta f_p \leq 500$ MPa.

A6.2 Axis and width of tie

The centroid of the reinforcement in a tie shall coincide with the axis of the tie in the strut-and-tie model.

The assumed tie width shall account for the distribution of reinforcement, the available cover to the surface of the reinforcement and the dimensions of the nodes at the ends of the tie.

A6.3 Anchoring of tie reinforcement

A6.3.1 *By mechanical devices*

Tie reinforcement shall be anchored by mechanical devices, post-tensioning anchorage devices, standard hooks, or straight bar development as required by A6.3.2 to A6.3.5.

A6.3.2 *Force developed in nodal zone*

Nodal zones shall develop the difference between the tie force on one side of the node and the tie force on the other side.

A6.3.3 *Point of application of tie force for one tie*

At nodal zones anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the nodal zone and enters the span.

A6.3.4 *Point of application of tie force for two or more ties*

At nodal zones anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the nodal zone.

A6.3.5 *Anchoring transverse reinforcement*

The transverse reinforcement required by A5.3 shall be anchored in accordance with 8.6.3.

A6.4 *Tie force where bar development limited*

Where the size of a nodal zone is not large enough to allow the yield strength of the reinforcement to be developed, the amount of reinforcement provided should be based on reduced design tensile stresses, f_s, which can be developed in the given nodal zone.

A7 Strength of nodal zones

A7.1 *Nominal compression strength*

The nominal compression strength of a nodal zone shall be:

$F_{nn} = f_{cu}A_n$ ..... (Eq. A–7)

where f_{cu} is the effective compressive strength of the concrete in the nodal zone as given in A7.2 and A_n is (a) or (b):

(a) The area of the face of the nodal zone that F^* acts on, taken perpendicular to the line of action of F^*; or

(b) The area of a section through the nodal zone, taken perpendicular to the line of action of the resultant force on the section.

A7.2 *Compressive stress on face of nodal zone*

Unless confining reinforcement is provided within the nodal zone and its effect is supported by test and analysis, the calculated effective compressive stress on a face of a nodal zone due to the strut-and-tie forces shall not exceed the value given by:

$f_{cu} = \alpha_1 \beta_n f'_c$ ..... (Eq. A–8)

where $\alpha 1$ is given by 7.4.2.1 (c) and the value of β_n is given in (a) to (c) below as appropriate.

(a) In nodal zones bounded by struts of bearing areas, or both $\beta_n = 1.0$; or

(b) In nodal zones anchoring one tie $\beta_n = 0.80$; or

(c) In nodal zones anchoring two or more ties $\beta_n = 0.60$.

A7.3 *Nodal zones for three-dimensional strut-and-tie models*

In a three-dimensional strut-and-tie model, the area of each face of a nodal zone shall be equal to or greater than that given in A7.1, and the shape of each face of the nodal zones shall be similar to the shape of the projection of the end of the struts onto the corresponding faces of the nodal zones.

A8 Considerations of seismic actions

A8.1 *General*

The strut and tie method may be used to design an element, or part of an element, for energy dissipation under seismic conditions provided.

(a) Capacity design is applied to ensure the energy dissipation is confined to the selected ductile struts and/or ties;

(b) Struts, which may also be subjected to tension on other phases of the earthquake, are confined by longitudinal reinforcement and ties as required in 10.4.7.

(c) Any tie reinforcement in a yielding element is anchored within the extended nodal zone to sustain the overstrength capacity of the reinforcement.

(d) All other ties are proportioned to sustain the maximum action that may be induced when overstrength actions are sustained in the selected struts or ties.

(e) At nodes, see A8.3(e) below.

(f) Where the strut and tie system is such that under reversed loading yielding in an element occurs only in tension, the element shall be designed as a nominally ductile element.

A8.2 *Diaphragms modelled by strut and tie*

Strut-and-tie models are appropriate for the design and detailing of diaphragms that may have irregularly arranged penetrations. In satisfying the requirements of Section 13, attention needs to be paid to the mobilising of several internal load paths, each set being dependent on the direction of the reversible seismic forces. During seismic response it is preferable to inhibit yielding in diaphragms.

A8.3 *Openings in walls modelled by strut and tie*

Structural walls with irregular openings may be designed by the use of strut and tie models for ductile or limited ductile behaviour as set out below:

(a) An energy dissipating mechanism is selected and capacity design is used to ensure that yielding is confined to the selected struts or ties;

(b) Where seismic force reversals require ties in previous loading stages to act or struts, the reinforcement must be adequately restrained against buckling and the concrete adequately confined to sustain the required compressive strength;

(c) By means of a capacity design approach, the yielding of ties in which force reversal cannot occur, as in the case of stirrups in beams or columns, should be inhibited;

(d) Each nodal zone must be examined for the possible reversal of nodal forces. Concrete design compression stresses in elastic regions of such walls should be limited to those given in A5 based on the maximum possible number of ties that could develop at the node under consideration;

(e) At nodes, where as a result of ductility demands, the reinforcement in a tie member could have yielded, concrete design compression stresses should be limited to 50 % of the values given in A5.

For example the particularly severe situation which arises when the nodal zone anchors tie in two or more directions, the limitation should be f_{cu} = 0.5 x 0.85 x 0.60 = 0.26. Figure A.2 illustrates a situation where these severe limitations on concrete compression strength are warranted.

Reinforcement which can be subjected to reversed cyclic inelastic strains should be provided with lateral support, using appropriate transverse reinforcement to prevent premature bar buckling.

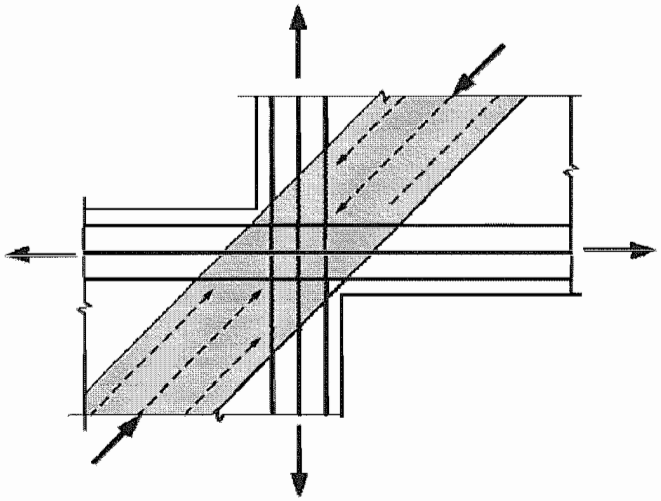


Figure A.2 – Typical nodal zone

APPENDIX B – SPECIAL PROVISIONS FOR THE SEISMIC DESIGN OF DUCTILE JOINTED PRECAST CONCRETE STRUCTURAL SYSTEMS (Normative)

B1 Notation

c	distance from the neutral axis to the extreme compression fibre, mm
d_{bl}	diameter of non-prestressed steel reinforcing bar, mm
E_{pt}	modulus of elasticity of prestressing tendons, MPa
$f_{pt,design}$	design upper limit for stress in prestressing tendons, MPa
$f_{pt,initial}$	initial stress in prestressing tendons, (after losses) MPa
f_{pty}	yield strength of prestressing tendons, MPa
f_y	yield strength of non-prestressed steel reinforcement, MPa
k_b	axial stiffness of one beam, N/mm
k_c	bending stiffness of one column at level of beam column connection, N/mm
k_{pt}	axial stiffness of post-tensioned tendons, N/mm
L_{cant}	distance from the column face to the point of contraflexure of the beam, mm
L_p	plastic hinge length of equivalent monolithic beam including strain penetration, mm
ℓ_{sp}	strain penetration of non-prestressed steel reinforcing bar, mm
ℓ_{ub}	unbonded length of prestressing tendons, mm
ℓ'_{ub}	unbonded length of non-prestressed reinforcing bar, mm
M	total moment capacity, N mm
M_N	flexural strength contribution due to axial load, N mm
M_{pt}	flexural strength contribution due to post-tensioned tensions, N mm
M_s	flexural strength contribution due to non-prestressed steel reinforcement, or energy dissipating devices, N mm
n	total number of joint openings at beam column interfaces along beam
S_p	structural performance factor
α_o	overstrength factor for non-prestressed steel reinforcement or energy dissipating device
Δ_{pt}	additional elongation at level of unbonded prestressing tendons due to gap opening, θ mm
Δ_s	elongation at level of non-prestressed steel reinforcement, mm
Δ_{sp}	elongation due to strain penetration of non-prestressed steel reinforcement, mm
ε_c	compressive strain in the concrete at the extreme fibre
$\varepsilon_{p,i}$	initial strain in unbonded post-tensioned prestressing
$\varepsilon_{pt}(\theta)$	additional strain in unbonded post-tensioned tendons due to gap opening θ
$\varepsilon_{pt,tot}$	total strain in unbonded post-tensioned tendons due to gap opening θ
$\varepsilon_s(\theta)$	strain in non-prestressed steel reinforcement due to gap opening θ
ε_u	ultimate strain of non-prestressed steel reinforcement
ε_y	yield strain of non-prestressed steel reinforcement
θ	rotation of gap opening
λ	moment contribution ratio between self-centering and energy dissipation
μ	structural ductility factor
ξ	viscous damping coefficient, %
ξ_{hybrid}	equivalent viscous damping of an hybrid connection/system
ξ_{lower}, ξ_{upper}	lower and upper bound values for equivalent viscous damping
ϕ_u	ultimate curvature of beam
ϕ_y	yield curvature of beam
Ω	factor indicating restraint effects to beam elongation

B2 Definitions

B2.1 *Jointed systems*

Jointed systems are structural systems in which the connections between the precast concrete elements are weaker than the elements themselves. Jointed systems do not emulate cast-in-place concrete construction. The connections of jointed systems can be of limited ductility or ductile.

B2.2 *Hybrid systems*

Hybrid systems are jointed structural systems in which the self-centering capability is provided by post-tensioning and/or axial compressive load, and energy dissipation is provided by yielding non-prestressed steel reinforcement or other special devices. Hybrid systems are ductile.

B2.3 *Equivalent monolithic systems*

Equivalent monolithic systems are structural systems in which the connections between the precast concrete elements are designed to emulate the performance of cast-in-place concrete construction. The connections can be of limited ductility or ductile.

B3 Scope and limitations

This Appendix applies to ductile jointed and hybrid precast concrete structural systems. The systems may be moment resisting frames, structural walls or dual systems, in which the precast concrete elements are joined together by post-tensioning techniques with or without the presence of non-prestressed steel reinforcement or other energy dissipating devices.

B4 General design approach

B4.1 *General*

Either a force-based or a displacement-based design approach shall be used for the seismic design of jointed and hybrid structural systems. Modifications to the inter-storey drift limits used in design shall be made in accordance with B4.2.

B4.2 *Drift limits*

Inter-storey drift limits as defined in NZS 1170.5 shall be adopted for jointed structures, except that drift limits corresponding to a damage control, or the serviceability limit state may be increased by up to 50 %, provided analytical calculations and/or experimental validation demonstrates a reduced level of damage, (both structural and non-structural), when compared to an equivalent monolithic structure. No increase in drift limit corresponding to the ultimate limit state shall be allowed where high inelastic demand and P-delta effects can govern the response.

B4.3 *Self-centering and energy dissipation capabilities of hybrid structures*

B4.3.1 *Combination of self-centering and energy dissipation*

An adequate combination of self-centering and energy dissipation contributions of a hybrid structural system shall be provided as specified in B4.3.2 and B4.3.3 in order to stay within maximum drift limits as well as avoiding residual deformations.

B4.3.2 *Condition for full self-centering*

The full self-centering of a general jointed connection shall be achieved by selecting, in the design phase, an appropriate moment contribution ratio λ as follows:

$$\lambda = \frac{M_{pt} + M_N}{M_s} \geq \alpha_o \quad \text{(Eq. B–1)}$$

where M_{pt}, M_N, and M_s are the flexural strength contributions of the post-tensioned tendons, the axial load where present, and the non-prestressed steel reinforcement or energy dissipating devices calculated with respect to the centroid of the concrete compression resultant of the section, respectively, and α_o (≥ 1.15) is the overstrength factor for the non-prestressed steel reinforcement or the energy dissipating devices.

B4.3.3 *Evaluation of energy dissipating capacity*

The energy dissipation capacity provided by the flag-shape hysteresis rule typical of hybrid systems, shall be evaluated in terms of the equivalent viscous damping percentage, ξ, by interpolation between a pure dissipative system (that is equivalent monolithic system with elasto-plastic or near elasto-plastic behaviour) and a pure self-centering system (that is an unbonded post-tensioned system with a non-linear elastic behaviour).

B4.3.4 *Structural performance factor, S_p*

Values of the S_p factor, used to evaluate the input seismic loads according to a force-based design approach, shall be consistent with the values adopted for ductile structures from 2.6.2.2.

B5 Behaviour of connections

B5.1 *Inelastic behaviour of connections*

In jointed structural systems the inelastic demand shall be concentrated within the critical connections between the precast concrete elements as a result of the opening and closing of a crack at the interface between elements.

B5.2 *Behaviour of unbonded post-tensioned tendons*

The unbonded post-tensioned tendons in the precast concrete elements which cross the interfaces between elements shall be designed to remain in the elastic range during the design earthquake.

B5.3 *Hybrid systems*

In hybrid systems in addition to unbonded post-tensioned tendons there shall be present at the critical connections non-prestressed steel reinforcement or other means of energy dissipation.

B5.4 *Shear transfer at critical connections*

B5.4.1 *Means of shear transfer at connections*

Vertical shear forces shall be transferred at the critical connections by shear keys, concrete corbels, metallic corbels or other means for which there is experimental or analytical evidence of satisfactory performance.

B5.4.2 *Shear transfer by friction induced by tendons*

Friction induced by post-tensioned tendons shall not be used in design to transfer shear at critical connections due to gravity loads. Transfer of shear force due to the seismic loads may rely on friction at the interface induced by the tendons provided the tendons remain in the elastic range at the design level of inter-storey drift of the structure.

B5.4.3 *Minimum shear capacity of connections*

The minimum shear capacity provided in accordance with B5.4.1 shall be in any case at least equal to the design shear force due to the factored gravity loads.

B5.4.4 *Torsion transfer at critical connections*

Torsion forces in the beam, i.e. due to the weight of a floor system orthogonal to the primary frame, shall be transferred at the critical connections by means of appropriate shear keys, concrete or metallic corbels/brackets, dowel action of the non-prestressed reinforcement or other means for which there is experimental or analytical evidence of satisfactory performance.

B6 Design of moment resisting frames

B6.1 *General*

Design of beams, columns and beam column joints shall satisfy the requirements of 19.4.3, 19.4.4 and 19.4.5, respectively. In particular, capacity design principles apply to achieve a desired beam sway, weak-beam strong-column inelastic mechanism.

B6.2 *Anchorage, location and longitudinal profile of the post-tensioned tendons*

B6.2.1 *Post-tensioning materials*

Strands or bars may be used for post-tensioned tendons provided that there is compliance with the limits on strain and stress included in this Appendix.

B6.2.2 *Loading cycles for anchorages*

Anchorages shall withstand, without failure, a minimum of 50 cycles of a loading for which the load in each cycle is varied between 40 % and 80 % of the minimum specified strength of the prestressing tendons.

B6.2.3 *Profile of post-tensioned tendons*

The profile of the post-tensioned tendons may be either straight or draped to follow the bending moment diagram.

B6.2.4 *Length of unbonded post-tensioned tendons*

No limit on the length of the unbonded post-tensioned tendons is required. The unbonded length shall be clearly defined and controlled as per design requirements.

B6.2.5 *Location of tendons at joint*

The location of the tendons in beams in the critical section where the gap may open and close can vary according to the design requirements. A concentric location in the beam is however preferred for seismic resisting systems in a high seismic region for an easier control of the additional elongation under lateral loading.

B6.3 *Prestressing force in beams*

B6.3.1 *Lower and upper bound for initial prestress*

The initial prestress shall be limited by a lower bound, in order to guarantee sufficient moment contribution M_{pt} to provide full self-centering capacity in accordance with B4.3.2 as well as by an upper bound in order for the tendons to remain in the elastic range for a target inter-storey drift level, while still providing self-centering properties.

B6.3.2 *Upper bound for initial prestress*

The condition for the upper bound limit for the initial prestress in force shall be expressed as:

$$f_{pt,initial} \leq 0.9\, f_{pty} - E_{pt}\varepsilon_{pt} \quad \text{(Eq. B–2)}$$

where

$f_{pt,initial}$ is the initial prestress, after losses

E_{pt} is the modulus of elasticity of the tendons

ε_{pt} is the additional strain in the tendons due to the lateral drift/displacement, calculated as per B6.4.6; and

f_{pty} is the yield strength of prestressed tendons

B6.4 *Evaluation of flexural strength at target inter-storey drift levels*

B6.4.1 *Evaluation of nominal flexural strength*

The evaluation of the nominal flexural strength at a target inter-storey drift value shall be according to Sections 7, 9 and 10 with account taken of the special conditions of equilibrium and member compatibility in accordance with B6.4.2 to B6.4.11.

B6.4.2 *Strain compatibility not applicable*

Due to the presence of unbonded tendons compatibility between strain in the tendons and strain in the concrete does not apply at any given section.

B6.4.3 *Member compatibility and equilibrium*

Member compatibility and section equilibrium conditions shall be satisfied.

B6.4.4 *Evaluation of strain in non-prestressed steel reinforcement*

The evaluation of the strain in the non-prestressed steel reinforcement or additional energy dissipation devices shall rely on section compatibility considerations only if fully bonded conditions are present. If

partial debonding exists then B6.4.7 or a similar approach properly validated by experimental tests shall be followed.

B6.4.5 *Evaluation of additional elongation of the unbonded tendons or bonded non-prestressed steel reinforcement*

The gap opening (rotation θ) mechanism shall be used to evaluate the additional elongation of the tendons, Δ_{pt}, and of the non-prestressed steel reinforcement or energy dissipation devices, Δ_s, which are assumed to be directly proportional to the distance from the neutral axis.

B6.4.6 *Strain in unbonded post-tensioned tendons*

The strain level in the unbonded post-tensioned tendons $\varepsilon_{pt}(\theta)$ due to the gap opening, θ, shall be calculated as:

$$\varepsilon_{pt}(\theta) = \frac{n\Delta_{pt}}{\ell_{ub}} \quad \text{(Eq. B–3)}$$

where n is the total number of joint openings at beam column interfaces along the beam involving the tendons, and ℓ_{ub} is the unbonded length in the tendons.

The total strain in the unbonded post-tensioned tendons $\varepsilon_{pt,tot}$ at a given gap rotation θ shall be given by the sum of the initial strain due to the prestress and the additional strain due to the gap opening given in B6.4.6. That is:

$$\varepsilon_{pt,tot} = \varepsilon_{p,i} + \varepsilon_{pt}(\phi) \quad \text{(Eq. B–4)}$$

B6.4.7 *Strain in unbonded non-prestressed longitudinal steel reinforcement*

When energy dissipation is assigned to longitudinal non-prestressed reinforcement, a defined debonded length, ℓ'_{ub}, can be deliberately adopted in the beam adjacent to the interface in order to avoid premature fracture of the reinforcement. In these conditions, section compatibility does not apply and the strain in the steel $\varepsilon_s(\theta)$ due to the gap opening θ shall be evaluated as:

$$\varepsilon_s(\theta) = \frac{(\Delta_s - 2\Delta_{sp})}{\ell'_{ub}} \quad \text{(Eq. B–5)}$$

where Δ_{sp} is the contribution to the gap opening due to the strain penetration of the non-prestressed steel reinforcement assumed to occur at both ends of the small unbonded region, ℓ'_{ub}.

After simplifications an approximate formula that may be used is:

$$\varepsilon_s = \frac{(\Delta_s + 2/3\ell_{sp}\varepsilon_y)}{(\ell'_{ub} + 2\ell_{sp})} \quad \text{(Eq. B–6)}$$

where

ℓ_{sp} is the strain penetration taken as $0.022\, f_y d_{bl}$

f_y is the yield strength of reinforcement

d_{bl} is the diameter of the reinforcing bar

B6.4.8 *Maximum strain in non-prestressed steel reinforcement*

At design level inter-storey drift, the strain in the non-prestressed steel reinforcement or in the dissipation devices should not exceed 90 % of the ultimate deformation capacity; that is $\varepsilon_s(\theta) \le 0.9\, \varepsilon_u$.

B6.4.9 *Neutral axis position*

Evaluation of the neutral axis depth c as well as of the concrete compression strain ε_c corresponding to a given level of inter-storey drift or gap opening rotation may be obtained by an iterative procedure assuming member compatibility conditions as per B6.4.10.

B6.4.10 *Concrete compression strain*

The compressive strain in the concrete at the extreme fibre, ε_c, may be evaluated using the following expression, which satisfies member compatibility conditions:

$$\varepsilon_c = \left[\frac{(\theta L_{cant})}{\left(L_{cant} - \frac{L_p}{2}\right) L_p} + \phi_y \right] c \qquad \text{(Eq. B–7)}$$

where

c is the neutral axis depth

L_p is the plastic hinge length of an equivalent monolithic connection (including strain penetration component)

L_{cant} is the distance between the column interface and the point of contraflexure (length of the beam cantilever), and

ϕ_y is the yield curvature of the section in an equivalent monolithic connection

B6.4.11 *Evaluation of neutral axis position*

For use with B6.4.10, simplified design charts or tables with the position of the neutral axis at different limit states may be adopted, if based on analytical procedures validated through experimental results.

B6.5 *Cyclic moment behaviour and energy dissipation*

B6.5.1 *Hysteresis behaviour*

The cyclic moment-rotation behaviour of a generic hybrid connection shall be described by a flag-shape hysteretic rule given by the combination of a non-linear elastic rule with an energy dissipating rule (elasto-plastic; Ramberg-Osgood, or other stiffness degrading rule), representing the moment contributions of the post-tensioned tendons, M_{pt}, and of the mild steel or energy dissipation devices, M_s.

B6.5.2 *Flag-shaped hysteresis rule*

The properties of the flag-shape hysteresis rule depend on the ratio between the moment contributions, which shall be evaluated about the concrete compression force resultant.

B6.5.3 *Equivalent viscous damping*

The equivalent viscous damping of an hybrid connection/system, ξ_{hybrid}, depends on the ratio of the moment contributions (M_{pt}/M_s) and may be directly evaluated from the resultant flag-shape hysteresis rule or as interpolation between lower and upper bound values given by an unbonded connection (ξ_{lower} = 5 %) and a monolithic frame system ξ_{upper} (Equation B–8).

$$\xi = 5 + 30\left(1 - \frac{1}{\sqrt{\mu}}\right) \% \qquad \text{(Eq. B–8)}$$

where μ is the structural ductility factor.

B6.5.4 *Contact damping*

In addition to the hysteretic damping evaluated according to B6.5.3, contact (radiation) damping can also be taken into account, provided experimental evidence of the dynamic rocking behaviour of the connection/system is available.

B6.6 *Design of column-to-foundation connection*

B6.6.1 *Performance*

The column-to-foundation connection shall be designed according to the target performance, in order to provide satisfactory self-centering properties and energy dissipation capabilities to the whole frame system.

B6.6.2 *Design approach similar to beam column connection design*

A design approach similar to that derived for the beam column connections in B6 shall be followed with modifications based on case-by-case considerations.

B6.6.3 *Contribution of column axial load to self centering*

The self-centering contribution of the axial load in the column, due to the unfactored gravity axial load shall be taken into account in the definition of the global hysteresis behaviour and self-centering capacity.

B6.6.4 *Prevention of sliding*

Sliding at the column-to-foundation connection shall be prevented by using appropriate shear keys or by relying on dowel action of the non-prestressed steel reinforcement passing through the critical section and on the shear-friction capacity provided by the vertical axial force due to unfactored gravity loads.

B7 Design of structural wall systems

B7.1 *General*

The design of jointed structural wall systems shall follow the general approach indicated in the previous sections with modifications as in B7.2 to B7.7.

B7.2 *Energy dissipation devices*

Energy dissipation devices shall be internal or external, placed either at the base section or between coupled panels and relying on the relative vertical movement during the rocking motion of the wall.

B7.3 *Axial loads*

The contribution of the axial load in terms of strength and stiffness shall be taken into account using unfactored gravity loads.

B7.4 *Ratio between self-centering and energy dissipation contributions*

The ratio between the self-centering and energy dissipation contributions shall account for axial force due to gravity load as in B7.3.

B7.5 *Effects of seismic actions on axial force*

The effects of seismic actions on the axial force shall be considered whenever they lead to less desirable behaviour due to a reduction of energy dissipation or an increase in P-delta effects in each direction of seismic action.

B7.6 *Overstrength of energy dissipation device*

Overstrength of the energy dissipation device shall be accounted for in order to ensure a full self-centering capability.

B7.7 *Displacement incompatibility due to rocking*

Displacement incompatibility issues due to the rocking motion of the wall (uplifting) and involving the connection details with the diaphragm, shall be addressed as in B8.

B8 System displacement compatibility issues

B8.1 *Gravity load carrying systems*

Structural wall or frame systems, which are primarily assigned to carry gravity loads, shall be able to accommodate the global system displacements without reduction of their vertical load-bearing capacity. If yielding of secondary elements is expected due to the deformation induced by the primary systems,

allowance should be made in the amount of post-tensioning to overcome the inelastic behaviour and guarantee full re-centering capacity.

B8.2 *Non-structural elements*

The expected damage to non-structural elements shall be evaluated from the displacements of the structural system.

B8.3 *Diaphragms*

The effects on diaphragm action due to the interaction between floor systems and the lateral load resisting systems (B8.5) shall be taken into account. Both the horizontal and vertical relative displacement incompatibility shall be estimated and properly accommodated.

B8.4 *Beam elongation*

B8.4.1 *Effects*

When frame systems are adopted for lateral load resisting systems, the effects of beam elongation (increase of distance between column centrelines) shall be taken into account in terms of: damage to and interaction with the floor system, increase of column curvature and of flexural and shear demand, increase of beam moment capacity due to the beam axial force, and increase of residual local deformations.

B8.4.2 *Seating of precast floor units*

Seating details for precast concrete floors shall take into account the expected beam elongation effects.

B8.4.3 *Post-tensioning force*

The effects of beam-elongation on the increase or decrease of the strain in the post-tensioning reinforcement shall be evaluated.

B8.4.4 *Estimation of post-tensioning strain due to beam elongation*

A simplified estimation of the strain increase in the post-tensioning steel, ε_{pt}, due to beam-elongation effects within a frame system can be obtained multiplying Equation B–4 by a factor $\left(1-\frac{1}{\Omega}\right)$ as follows:

$$\varepsilon_{pt} = \varepsilon_{in} + \frac{n\Delta_{pt}}{\ell_{ub}}\left(1-\frac{1}{\Omega}\right) \quad \text{(Eq. B–9)}$$

where

Ω is an indicator of the restraint effects given in a two bays, three columns, one storey sub-assembly, by:

$$\Omega = \frac{k_b}{k_c + 2k_{pt}} + 1 \quad \text{(Eq. B–10)}$$

where

k_b is the axial stiffness of one beam

k_c is the bending stiffness of one column at the level of the beam-to-column connection, and

k_{pt} is the axial stiffness of the post-tensioned tendons spanning the entire subassembly

B8.5 *Floor-to-lateral-load resisting system incompatibility*

B8.5.1 *Relative vertical displacements incompatibility*

The expected relative vertical displacements and connection forces between lateral resisting systems and diaphragms shall be evaluated.

B8.5.2 *Connection details*

Special connection details able to accommodate the relative displacements and sustain the increased forces as estimated in B8.5.1 shall be adopted.

B8.5.3 *Location of connections*

In order to minimize additional relative displacements and forces, the connections shall be placed in regions of relatively limited vertical displacement incompatibility and dimensioned to accommodate the expected deformations without affecting their capacity to transfer the inertia forces to the lateral load resisting systems.

B8.5.4 *Design strength for the collectors*

The design of the floor-to-lateral resisting systems collectors shall account for the expected overstrength coming from a possible diaphragm inelastic response as well as for the increased floor acceleration values due to high mode effects.

B8.5.5 *Inelastic behaviour and energy dissipation of collectors*

The floor-to-lateral resisting systems connectors may be assigned an inelastic behaviour with energy dissipation, if adequate evidences of satisfactory global performance from analytical studies on the overall system (floor-lateral resisting system interaction) are provided.

NOTES

(There is no appendix C so as to avoid confusion with commentary clauses.)

NOTES

APPENDIX D – METHODS FOR THE EVALUATION OF ACTIONS IN DUCTILE AND LIMITED DUCTILE MULTI-STOREY FRAMES AND WALLS (Normative)

D1 Notation

D1.1 Standard symbols

A_g gross area of column section, mm^2

$E_{o,i}$ lateral force at level i when overstrength actions are sustained, N

f_c' specified compressive strength of concrete, MPa

h_c column depth, a section dimension, mm

I_e effective moment of inertia of a section, mm^4

k relative flexural stiffness, mm^3

L length of member between centrelines of supports, mm

L_n clear length of column between beam faces, mm

M_n nominal flexural strength, N mm

N_o^* capacity design axial load on a column, N

N_{oe} axial load in a column due to shear induced in a beam by end moments when overstrength moments act in the beam, N

R_m moment reduction factor for column under low axial compression or in axial tension

R_v axial load reduction factor

T_1 the computed period of the structure in its first mode of translational vibration, s

V_{col}^* capacity design column shear force, N

V_E shear force in a column found from an equivalent static of first mode analysis, N

V_{oe} shear force at the face of a column induced by end moments in a beam when overstrength moments act in the beam, N

β modification factor for dynamic magnification factor

ϕ strength reduction factor

ϕ_o overstrength factor for a joint zone or the base of a column

$\phi_{o,i}$ overstrength factor for lateral force at a level in a frame

$\phi_{o,fy}$ overstrength factor depending on reinforcement grade

ϕ_o' average overstrength factors for beam column joint zones located above and below the column being considered

ω dynamic magnification factor for bending moments

ω_{max} maximum value of ω which acts in mid-height region of a multi-storey frame

ΣM_{ob} beam input overstrength moment into a beam and column joint zone at intersection of centre-lines at intersection of centre-lines when overstrength moments act in primary plastic regions, N mm

ΣM_n sum of bending moments in beams sustained at the intersection of the beam and column centrelines when nominal moments act in the beams at the column faces, N mm

$M_{oc,bottom}$ Overstrength moment in a column at the bottom of the first storey, N mm

$M_{oc,top}$ Overstrength moment in a column at the top of the first storey, N mm

D2 General

This Appendix specifies methods of determining design actions for structural members and parts of members, which contain primary plastic hinge regions for situations where capacity design is required by 2.6.5 for:

(a) Ductile and limited ductile moment resisting frame structures;

(b) Ductile and limited ductile walls;

(c) Ductile and limited ductile dual wall frame structures.

As the design methods in this appendix are based on capacity design a strength reduction factor of 1 shall be used (see 2.6.5 and 2.3.2.2) for all calculations of member capacities .

D3 Columns multi-storey ductile frames

D3.1 *General*

Where not exempted 2.6.7.2 columns by in multi-storey ductile frames, shall be designed by either Method A or Method B as identified in the following clauses. Both methods give the structure a high level of protection against the formation of a storey column sway mechanism and consequently are suitable for capacity design.

Method A gives a high level of protection against the formation of plastic regions in columns between 3 h_c above the first storey and 3 h_c below the top storey.

Method B gives a lower level of protection against the formation of localised plastic regions. However, it still gives a high level of protection against the formation of a storey column sway mechanism.

Outline of Methods

In both methods:

(a) A suitable ductile failure mechanism shall be identified and the locations of associated primary plastic regions (hinges) shall be defined. Acceptable ductile failure mechanisms are identified in 2.6.7.2.

(b) The magnitudes of the bending moments and shear forces acting in the beams at the faces of the columns shall be found when overstrength moments act in the primary plastic regions.

(c) The magnitude of the total moment applied to a beam column joint at the intersection point of the beam and column centrelines shall be calculated. This moment is referred to as the beam input overstrength moment.

(d) The reinforcement in the columns in any storey need not exceed the amount required for a column designed as nominally ductile, with the strength reduction factor (ϕ) equal to 0.85 for actions derived from an analysis based on the assumption that the structure is nominally ductile and the S_p factor is 0.9. The requirements of 2.6.6.1 (b) shall apply to the structure.

(e) Different detailing provisions apply to Method A and Method B, and shall be applied as appropriate.

In Method A within the zone between 3 h_c above the first storey and 3 h_c below the top storey.:

(i) The position of lap splices in longitudinal reinforcement is not limited, see 10.4.6.8.2 (a) but outside of this zone this relaxation shall not be applied.

(ii) The quantity of confinement reinforcement may be reduced as specified in 10.4.7.4.3 and 10.4.7.5.3. Outside this zone this reduction shall not be applied.

In Method B the relaxation in the location of lap splices (in 10.4.7.4.3 (a)) and the quantity of confinement (10.4.7.4.3) shall not be applied.

The two methods do not apply to:

(a) Frames of two storeys or less in which column sidesway mechanisms are intended to form the primary energy dissipating mechanism as defined in 2.6.7.2;

(b) Frames in which column displacements are predominantly controlled by structural walls, as in wall frame (dual) structures;

(c) Columns that act as props, which are not required or intended to contribute to lateral force resistance.

A2 **Method A** is intended for use in the design of regular or near regular ductile moment resisting frames, in which most columns exhibit a point of inflection in each storey, except those near the base of the building, when subjected to first mode or equivalent static analysis actions.

Method B is a general approach, which may be used on a wider range of ductile moment resisting frame structures than Method A. There are no restrictions in terms of regularity. With this Method the designer has the freedom to locate potential plastic regions in some of the columns.

D3.2 Design moments and shears in columns by Method A

D3.2.1 *General*

Three steps are required to determine design moments in the columns at each beam column intersection as set out below:

(a) The beam input overstrength moment at each beam column joint shall be distributed into the columns above and below the intersection of the beam and column centre-lines as set out in D3.2.2.

(b) The column moments from step (a) shall be multiplied by the dynamic magnification factor, ω, and the modification factor, β, as set out in D3.2.3.

(c) The critical capacity design moments for the columns at the beam faces shall be calculated as set out in (i) and (ii) below;

  (i) The bending moments found from (a) and (b) above, which are the intersection point of the beam and column centrelines, shall be projected to the column at the face of the beam as detailed in D3.2.4.

  (ii) Where the axial load associated with capacity design conditions in a column is small, or it is subjected to tension in the critical load case, the design moment may be reduced as set out in D3.2.5.

D3.2.2 *Distribution of beam input moment into columns*

A2

Divide the frame being designed into two regions on the basis of an equivalent static or first mode analysis. Region 1 is located between the mid height of the top storey in the frame and the level in which the lowest point of inflection occurs in each column. Region 2 is located between the line connecting the lowest points of inflection in the columns and the mid height of the storey containing the primary plastic hinge (generally at the base of the columns). In both regions the actions acting on each beam column joint zone are considered individually.

Region 1

In this region, the moments from the equivalent static or first mode analysis in each column immediately above and below the centroid of the joint zone being considered are multiplied by ϕ_o, such that the sum of the two moments is equal to the sum of the over-strength moments which may be sustained by the beam at the centroid of the beam column joint zone.

Region 2

In this region, the moments sustained in the column immediately above and below the centre of the joint zone being considered are multiplied by the factor $\phi_{o,b}$, which is given by:

$$\phi_{o,b} = M_{o,base} / M_{E,base}$$

Where $M_{o,base}$ is the overstrength moment at the critical section of the primary plastic hinge in the column being considered and $M_{E,base}$ is the corresponding moment induced by the equivalent static or first mode actions.

Both regions

The required design strengths (M^*) of the critical section in the potential primary plastic hinge in the column (generally at the base of the column), and in the section immediately below the uppermost level, are found from the critical combination of ultimate strength actions including either wind or earthquake actions as specified in 2.3.2 and 2.6.7.5(d). The overstrength in the column primary plastic hinge is determined from the section details used to satisfy the design strength requirements, with the material strengths and moment amplification specified in 2.6.5. The overstrength moments at the critical sections in the beams are found as defined in 2.6.5 and 9.4.1.6.2.

D3.2.3 *Dynamic magnification and modification factors*

The moments in the columns at the level of the beam centre-line at each joint zone shall be multiplied by an appropriate dynamic magnification factor, ω, which is defined in (i) below, and an appropriate modification factor β which is defined in (ii) below. Limits on the product of $\omega\beta$ are given in (iii) while (iv) defines the appropriate values of dynamic magnification factor for columns, which are part of two or more frames.

(a) The maximum value of the dynamic magnification factor, ω_{max} is given by:

$\omega_{max} = 0.6T_1 + 0.85$ ..... (Eq. D–1)

but not less than 1.3 or more than 1.8.

The value of dynamic magnification factor, ω, varies over the height of the building. At the base of the columns and at the top of the upper storey the value of ω shall be taken as 1.0. Between 30 % of the height of the frame above the base and the third highest level in the frame, ω shall be taken as ω_{max}. The value of ω at the second level is the larger of 1.3 or that obtained by linear interpolation between the value at the base of the column, and ω_{max} at 30 % of the height of the building. For the second to top level the value is the larger of 1.3 or that obtained by linear interpolation between ω_{max} at the third highest level and 1.0 at the highest level.

(b) The capacity design moments in the columns at a beam column joint zone, found above, may be reduced by the modification factor, β, which is given by Equation D-2. The maximum value of β is 1.0 and this value shall be used at the base of the columns and in the top storey of the building. Else where the value of β is given by;

A2

$$\beta = \left(1.4 - \frac{\sum M_o^{'}}{2.5\phi_{o.fy}\sum M_n^{'}}\right) \quad \text{..... (Eq. D–2)}$$

Where $\phi_{o,fy}$ is defined in 2.6.5.6;

A2

and $\sum M_o^{'}$ and $\sum M_n^{'}$ are the sums of the beam overstrength and nominal strength moments respectively, acting at the column faces of the beam column joint being considered.

ΣM_n is the corresponding sum of the moments when the beams are sustaining their nominal strength moments.

(c) The following limits apply to the product of dynamic magnification factor and modification factor, $\omega\beta$. All levels except the base of the columns and the top storey of the building the product of the dynamic magnification factor and the modification factor, $\omega\beta$, shall be equal to or greater than 1.3.

In the top storey and at the base of the columns the minimum value of $\omega\beta$ shall be equal to or greater than 1.2.

(d) Where a column is part of more than one frame bi-axial actions are induced in the column and the capacity design actions shall be found by considering the actions arising from all the beams framing into it at the level being considered. The dynamic magnification and modification factors for the moments in the column for the first frame shall be as defined in (a) and (b) above. The corresponding dynamic magnification and modification factors ($\omega\beta$) for the simultaneous actions from the second or subsequent frames shall be taken as 1.0. Where the enclosed angle between two frames is less than 45° the dynamic magnification for the two frames shall be given the same dynamic magnification and modification factors.

D3.2.4 *Critical design moments in columns*

The critical design moments for a column are at the level of the top and bottom faces of the beams. These critical values shall be found by calculating to moment at a beam face from the value found at the level of the beam centre-line together with the assumption that the shear force is equal to 60% of the capacity design shear force in the column, which is defined in D3.2.6.

D3.2.5 *Reduction in design moments for cases of small axial compression*

Where a column is subjected to small axial compression or net axial tension, in the critical load case, some reduction in the column design moments may be made. The minimum reduced column design moment at the face of the column may be found by multiplying the design moment found in D3.2.4 by the reduction factor, R_m given in Table D.1. The axial load level required in this table is defined in D3.4.

Table D.1 – Moment reduction factor R_m

	Tension							Compression			
$\frac{N_o^*}{f_c' A_g}$	≤ –0.150	–0.125	–0.100	–0.075	–0.050	–0.025	0.00	0.025	0.050	0.075	≥ 0.10
ω											
1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.1	0.85	0.86	0.88	0.89	0.91	0.92	0.94	0.95	0.97	0.98	1.00
1.2	0.72	0.75	0.78	0.81	0.83	0.86	0.89	0.92	0.94	0.97	1.00
1.3	0.62	0.65	0.69	0.73	0.77	0.81	0.85	0.88	0.92	0.96	1.00
1.4	0.52	0.57	0.62	0.67	0.71	0.76	0.81	0.86	0.90	0.95	1.00
1.5	0.44	0.50	0.56	0.61	0.67	0.72	0.76	0.83	0.89	0.94	1.00
1.6	0.37	0.44	0.50	0.56	0.62	0.69	0.75	0.81	0.88	0.94	1.00
1.7	0.31	0.38	0.45	0.52	0.59	0.66	0.73	0.79	0.86	0.93	1.00
1.8	0.30	0.33	0.41	0.48	0.56	0.63	0.70	0.78	0.85	0.93	1.00

NOTE – ω is the local value of the dynamic magnification factor applicable to the design of the column section at that level.

A2

D3.2.6 *Design shears in columns*

The design shear force in a column for seismic actions along an axis, V_{col}^*, shall be taken as the appropriate value given in (a) or (b) below, but in no case shall it be less than 1.6 times the shear force induced by the seismic design forces.

(a) In first storey columns the capacity design shear forces shall be equal to or greater than:

$$V_{col}^* = 1.15\,(M_{oc,bottom} + M_{oc,top}) / L_n \quad \text{(Eq. D-3)}$$

where $M_{oc,bottom}$ and $M_{oc,top}$ are the overstrength bending moments at the bottom and top of the column in the first storey and L_n is the clear height of the column in the storey. In calculating $M_{oc,bottom}$ allowance shall be made for the increase in strength arising from confinement of the plastic hinge region by any foundation beam or pad as required in 2.6.5.5(b).

(b) In columns above the first storey and excluding the top storey, V_{col}^*, shall be given by:

$$V_{col}^* = 1.3\,\phi_o'\,V_E \quad \text{(Eq. D–4)}$$

Where V_E is the shear in the column being considered found from an equivalent static or first mode analysis for seismic actions and ϕ_o' is the average overstrength factor for the beam column intersections for each end of the individual column in the storey being considered.

(c) In the top storey, where the column is expected to form a plastic region before the beam, Equation D-3 shall be used to find V_{col}^*. Where this condition is not met Equation D–4 shall be used

(d) In columns, which intersect with beams on two or more axes, the simultaneous action of the shear forces applied by the beams on each axis shall be considered in the design for shear in the column.

D3.2.7 *Design of columns*

The columns shall be designed to sustain simultaneously the critical combinations of capacity design axial forces as set out in D3.4, design bending moments, as set out in D3.2.4 and D3.2.5 and design shear forces as set out in D3.2.6.

D3.3 Design moments and shears in columns by Method B

D3.3.1 *General*

This method of assessing the capacity design actions in the columns involves the following steps for each level of the frame:

(a) Establish the location of the assumed points of inflection in the columns for Method B, as set out in D3.3.2

(b) Determine the beam input overstrength moment into each beam column on each joint in the level being considered, as set out in D3.1;

(c) Scale the lateral force acting at a level from an equivalent static or a first mode analysis so that it is consistent with overstrength actions being sustained in the beams in the level being considered. Distribute this lateral force to each of the beam column joints in the level, as set out in D3.3.3 and D3.3.8;

(d) Select the points of inflection in the storeys above and below the level being considered. From these determine the resultant shears in each column due to the beam input overstrength moment and the lateral forces acting on each of the beam column joint zones, as set out in D3.3.4;

(e) Multiply the column shears found in (d) by appropriate dynamic magnification and modification factors as set out in D3.3.5;

(f) Find the design moments at the critical sections of the columns, as set out in 3.3.6;

(g) Determine the critical design axial load acting in each storey of the column as set out in D3.4

(h) Proportion the column to sustain the critical combinations of moment, shear and axial load, as set out in D3.5.

D3.3.2 *Location of points of inflection in columns*

In **Method B** the locations of the points of inflection in the columns are assumed.

(a) Where an equivalent static or first mode analysis indicates that a point of inflection occurs in a storey, the points of inflection assumed for Method B, may be located at any convenient location within the mid half of the storey. Generally the most convenient location is at the mid-height of the storey.

(b) Where an equivalent static or first mode analysis indicates that the points of infection are not within the storey height, calculations shall demonstrate that the columns have sufficient ductility to accommodate the inelastic deformation associated with the chosen location of points of the inflection. In this case it may be necessary to assume some other location for the point of inflection than at the mid-height of the storey.

D3.3.3 *Lateral seismic forces at a level*

(a) A lateral force corresponding to overstrength actions, $E_{o,i,}$ acting at the level being considered in a frame, shall be found by scaling values from an equivalent static or first mode analysis by the factor, $\phi_{ol,i}$. The numerical value of $\phi_{ol,i}$ is equal to the ratio of the sum of the beam input overstrength moments in the level to the corresponding sum of bending moments from the equivalent static or first mode analysis.

Where the chosen ductile failure mechanism includes a primary plastic hinge forming in a column the sum of the beam input moments at a joint, ΣM_{ob}, may be taken as the sum of the moments at the intersection of the beam and column centre-lines applied to the joint zone by the columns, when overstrength moments act in primary plastic hinge regions.

(b) At all levels except the top, the lateral force is distributed to the individual beam column joint zones in the level being considered. There is considerable freedom as to how this lateral force, $E_{o,i,}$ is distributed to each joint zone, though the distribution must be such that the requirements of D3.3.8 are satisfied.

(c) The shear force induced in the columns above and below the joint being considered, due to the portion of the lateral force $E_{o,i}$ applied to that joint, is calculated from statics assuming;

(i) The component of lateral force acts as a point load on the column at the height of the beam centre-line;

(ii) The column is supported by shear forces acting at the level of the selected points of inflection in the storeys above and below the level being considered;

(iii) No bending moments are transferred from the columns to the beams

D3.3.4 *Column shear due to beam overstrength moments*

Each beam column joint at each level shall be considered in turn to determine the shear forces induced in the columns above and below the level being considered due to the beam input overstrength moment. These shear forces shall be calculated from statics assuming:

(a) The beam input overstrength moment acts at the intersection point of the beam, and column centre-lines;
(b) No lateral force acts on the joint zone;
(c) The columns are supported by lateral shear forces acting at the assumed points of inflection in the storeys above and below the level being considered.

D3.3.5 *Resultant column shears*

The resultant shear force in each column shall be taken as the sum of the shear forces due to the component of the overstrength lateral force acting the joint zone being considered, as set out in D3.3.3, and the corresponding shear force due to the input beam overstrength moment, as set out in D3.3.4. The sum of these two values shall be multiplied by the appropriate dynamic magnification and modification factors ($\omega\beta$) as set out in (a), (b) and (c) below.

(a) The dynamic magnification factor, ω, shall be taken as not less than;
  (i) 1.3 for all storeys except the top two;
  (ii) 1.15 for the second to highest storey;
  (iii) 1.0 for the highest storey.

(b) The modification factor β except at the base or in the top storey is given by:

$$\beta = \left(1.25 - \frac{\sum M_o'}{4.0\phi_{o.fy}\sum M_n'}\right) \quad \text{(Eq. D–5)} \qquad \text{A2}$$

The maximum value of β is 1.0 and this value, shall be used at the base of the columns and in the top storey of the building.

and $\sum M_o'$ and $\sum M_n'$ are the sums of the beam overstrength and nominal strength moments respectively, acting at the column faces of all the beam column joints of the frame in the level being considered. A2

$\phi_{o,fy}$ is defined in 2.6.5.6;

$\sum M_n'$ is the sum of the bending moments in the beams at the column faces in the level being considered when nominal moments act at the critical sections of all the potential plastic regions. A2

(c) The following limits apply to the β_w values:
In all storeys except the top two $\beta\omega \geq 1.2$
In the second to top storey $\beta\omega \geq 1.1$
In the top storey $\beta\omega \geq 1.0$.

(d) Where a column is part of more than one frame bi-axial actions are induced in the column and the capacity design actions shall be found by considering the actions arising from all the beams framing into it at the level being considered. The dynamic magnification and modification factors for the shear force in the column from the first frame shall be as defined in (a) and (b) above. The corresponding dynamic magnification and modification factors ($\omega\beta$) for the simultaneous actions from the second or subsequent frames shall be taken as 1.0. Where the enclosed angle between two frames is less than 45° the same dynamic magnification and modification factors shall be used for the two frames.

D3.3.6 *Capacity design column moments*

The capacity design moments in the columns at the face of the beams shall be taken as the product of the shear in the column, found in step, D3.3.5 times the distance between the point of inflection and the face of the beam. The calculated shear in a column is in general found twice, as in the calculations with Method B the shears are found for the columns above and below each level. The critical shear in any storey shall be taken as the maximum of these two values.

D3.3.7 *Design shear strength for columns*

For all columns, except those in the first storey, the capacity design shear strength shall be equal to or greater than a shear force equal to 1.15 times value found by dividing the sum of the nominal column moment strengths in the critical sections at the top and bottom of the storey by the distance between these critical sections.

In the first storey columns the minimum capacity design shear strength shall be equal to or greater than by:

$$V^*_{col} = 1.15\ (M_{oc,bottom} + M_{oc,top}) / L_n \quad \text{(Eq. D–6)}$$

where $M_{oc,bottom}$ and $M_{oc,top}$ are the overstrength bending moments at the top and bottom of the column in the first storey and L_n is the clear height of the column in the storey. In calculating $M_{oc,bottom}$ allowance shall be made for the increase in strength arising from confinement of the plastic hinge region by any foundation beam or pad as required in 2.6.5.5 (b).

D3.3.8 *Limit on distribution of column shear forces*

The distribution of the lateral force, $E_{o,i}$, to the joint zones in step D3.3.3 (ii) shall be such that the primary plastic hinge regions maintain their chosen locations when the structure is pushed laterally into the inelastic range.

D3.4 Capacity design axial forces for Methods A and B

The design axial forces in the columns shall be based on the assumption that the structure sustains dead load and long-term live load and that overstrength actions are sustained in all the primary plastic regions in the structure.

The component of axial force in a column, N_{oe}, which is due to the shear induced in the beams from the end moments (ΣV_{oe}) when overstrength moments act, may be reduced such that:

$$N_{oe} = R_v\ \Sigma V_{oe} \quad \text{(Eq. D–7)}$$

where R_v is a coefficient given by the expression:

$$R_v = 1.0 - 0.015\ n \geq 0.70 \quad \text{(Eq. D–8)}$$

n is the number of storeys above the level being considered,
ΣV_{oe} is the sum of the component of the shears in the beams due to the end moments, which are sustained when overstrength actions act in the beams.

D3.5 Design of columns

In all cases all sections of a column shall be designed to satisfy the minimum requirements of both the ultimate limit state and of capacity design actions (as set out in this Appendix). Where a column is incorporated in more than one frame it shall be designed to sustain the simultaneous actions transferred to it by the beams in all the frames connected to the column.

D4 Ductile and limited ductile walls

D4.1 General

A ductile failure mechanism, which is consistent with 2.6.5.2 and 2.6.8.1, shall be identified.

D4.2 Design moment envelope

The envelope for bending moments obtained from an equivalent static or modal analysis shall be;

(a) Multiplied by the ratio of the flexural overstrength moment sustained in the primary plastic region to the corresponding design action at this section
(b) Modified to allow for higher mode effects.

A recommended envelope for uniform walls with a near uniform distribution of seismic mass over the height of the wall is given in the commentary.

Longitudinal reinforcement in the wall shall, except at the top of the wall, be extended for a distance of the wall length (ℓ_w) plus a development length for the bar beyond the envelope.

D4.3 Design shear force envelope

The design shear force envelope for a structural wall shall be obtained from an equivalent static analysis by:

(a) Multiplying the shear force at each level by the ratio of the flexural overstrength to the design flexural action due to seismic forces at the critical section of the primary plastic region;

(b) Modifying the shear force envelope to allow for higher mode effects.

A suitable shear force envelope is given in the Commentary for the case of a uniform wall with a near uniform distribution of seismic weights at each level.

D4.4 Walls which are not uniform

Where walls are not uniform a rational method of design shall be used based on the concepts in this APPENDIX and 2.6.5.

D5 Wall-frame structures – Ductile and limited ductile

D5.1 General

Buildings in which the lateral resistance is provided by both walls and frames (dual structures) may be designed as a dual system as set out in Chapter 6 of Reference D.3 provided that:

The structure when analysed by the equivalent static method or by the first mode response in a modal analysis, each wall does not have more than one point of inflection above the point of maximum moment in the wall, and at some point within the lower third of the building the walls in the structure, the sum of the shear forces in all walls exceeds 30 % of the storey shear force.

In such structures the walls can be used to prevent the formation of column sway and mixed column beam sway modes from developing. This gives more freedom to the location of plastic regions in the beams and columns of the frame and it reduces the magnitude of dynamic amplification factors for columns.

Structures falling outside these criteria shall be the subject of rational design based on the concepts in contained in this appendix and 2.6.5.

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